FY '91 Task 14

Final Product
VA Coastal Resources Mgt. Program
12/31/92

Drainfield Repair Resource Manual

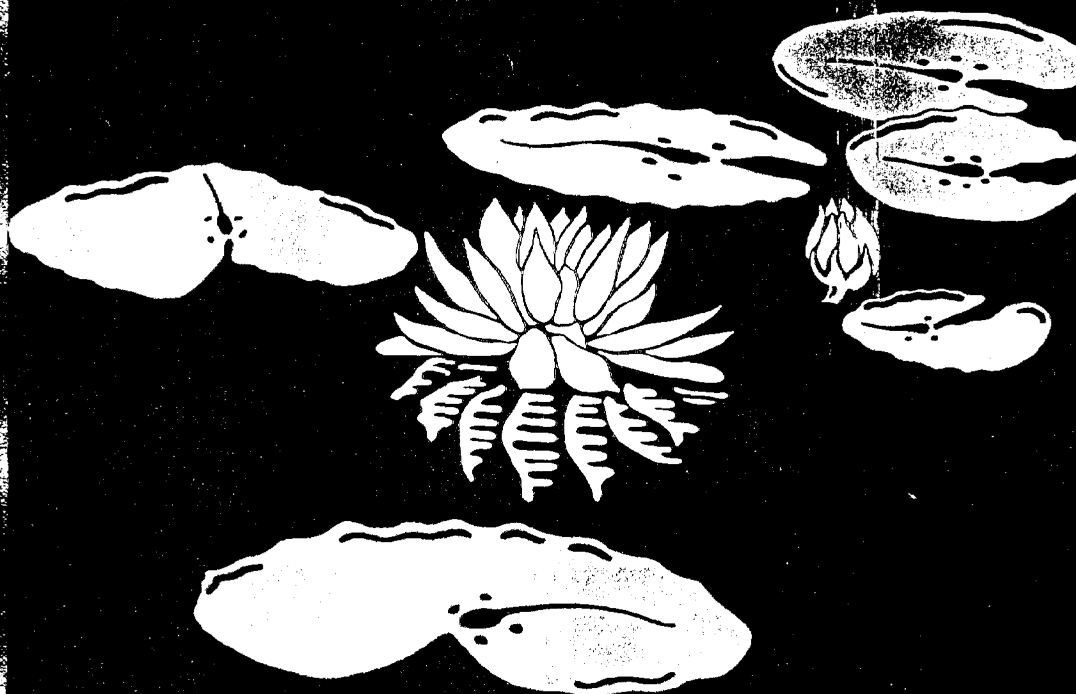
For Systems Failing in the Coastal Zone Area of Virginia

Produced by the Virginia Department of Health,
Division of Onsite Sewage and Water Services
Through a Grant Funded by NOAA and
Administered by the Council on the Environment

Constructed Wetlands for Wastewater Treatment

Municipal, Industrial and Agricultural

Donald A. Hammer
Editor



Class Outline
Drainfield Repair Technology
October 26-27, 1992 (Virginia Beach)
October 27-28 (Newport News)

Class 1

October 26, 1992
Virginia Beach Central Library

9:00 - Noon	High Pressure Drip Irrigation Design and Application Instructor: Tom Sinclair
Noon - 1:30	Lunch
1:30 - 4:30	Recirculating Sand Filters Design and Application Instructor: Rich Piluk

October 27, 1992
Virginia Beach Health Department

9:00 - 11:15	Constructed Wetlands and Upland-Wetland systems. Application, design and use in North Carolina Halford House
11:15 - Noon	Constructed Wetlands, Virginia's Experimental Program Experience

Class 2

October 27, 1992
Newport News Health Department

1:30 - 4:30 p.m.	High Pressure Drip Irrigation Design and Application Instructor: Tom Sinclair

October 28, 1992
Newport News Health Department

9:00 - Noon	Recirculating Sand Filters Design and Application Instructor: Rich Piluk
1:30 - 3:45 p.m.	Constructed Wetlands and Upland-Wetland systems. Application, design and use in North Carolina Halford House
3:45 - 4:30 p.m.	Constructed Wetlands, Virginia's Experimental Program Experience

Class Outline
Theory of Drainfield Failure and Repair

October 19, 1992 (Virginia Beach)
October 20, 1992 (Newport News)
October 21, 1992 (Glenns Campus)

9:30 - 9:45 a.m.	Introduction & Welcome
9:45 - 10:45 a.m.	Overview of why systems fail Instructor: Alexander
10:45 - 11:00 a.m.	Break
11:00 - Noon	Systematic approach to evaluating the cause(s) of a failing system. Instructor: Jones
Noon - 1:00 p.m.	Lunch
1:00 - 2:30 p.m.	Repair strategies to correct failing systems Instructor: Sandman
2:30 - 2:45 p.m.	Break
2:45 - 4:00 p.m.	Repair options and matching technology to solutions. The most expensive repair isn't necessarily the best. Instructors: Alexander and Sandman
4:00 - 4:30 p.m.	Wrap-up. Questions, answers and general discussion.

This training class was funded, in part, by the Virginia Council on the Environment's Coastal Resources Management Program through Grant #NA17OZ0359-01 of the National Oceanic and Atmospheric Administration, Office of Ocean and Coastal Resource Management, under the Coastal Zone Management Act of 1972 as amended

Repair Training Expectations

- Improve understanding about why systems fail
- Improve ability to I.D. causes
- Provide basis for designing an appropriate repair
- Generate internal interest
- Instill public confidence
    - * what helps?
    - * what hurts?

Class Perspective

- Informal (ask questions)
- Stimulate discussion
- Encourage creativity
- Encourage long term innovative thinking
- Variation between Specialists
- Variation in time

Introduction

- Treatment
- Biological Components
  - Bacteria
  - Viruses
- Physical and Chemical Components
  - BOD
  - SS
  - Nitrates
  - Other
- Disposal

Causes of Failure

* Hydraulic Failure *

- Loading rates
- Miscellaneous water sources
- Leaking fixtures
- Uneven distribution

Causes of Failure

* Physical Causes *

- Tree roots
- Age
- Material failure
  - * construction damage
  - * settling
- Soil clogging
  - * creeping failure
  - * mineralogy changes

Causes of Failure

* Landscape Position *

- Infiltration
- Off-site drainage

Causes of Failure
* Problem Soils *

- High water tables
- Slow infiltration rates
- Plastic clays
- Restrictive horizons

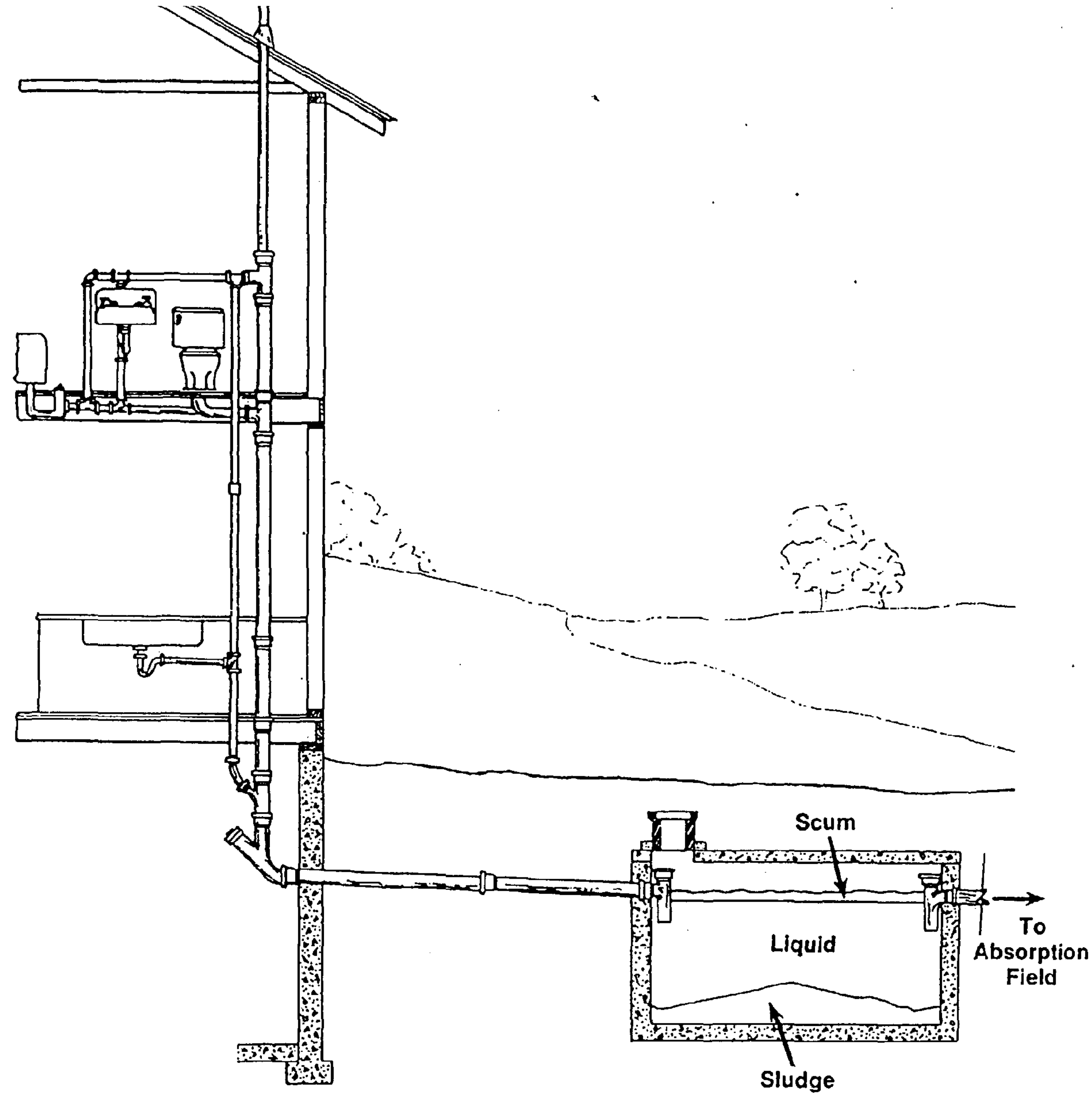
A Systematic Evaluation
* The House and Plumbing *

- Introduction
  logical approach
  keys solution to problem
  requires thought on each site
  (no cookbook solutions)

- Leaking fixtures

- Obstructed vent

- Obstructed sewer


Scum
Liquid
Sludge
To
Absorption
Field

A Systematic Evaluation
* The Septic Tank *

- Maintenance
  - When pumped
  - Observations when pumped
- Records and history
  - This system
  - Neighboring systems
- Tees
  - Present?
  - Condition
  - Materials
- Effluent level

A Systematic Evaluation

* Distribution Box *

- Condition and materials
  - Tree roots
  - Timing
  - Presence of sludge
- Effluent level
- Relative elevation to
  - Septic tank outlet
  - Drainfield

A Systematic Evaluation

* The Drainfield *

- System age and size
- Surface drainage
- Lines saturated?
- Evidence of physical damage
- Clogging mat present?
- Depth to limiting factor
  - water table
  - impervious horizon
  - plastic soils

TRADITIONAL SUBSURFACE SEEPAGE BED:

Gravity flow; continuous trickle of effluent.

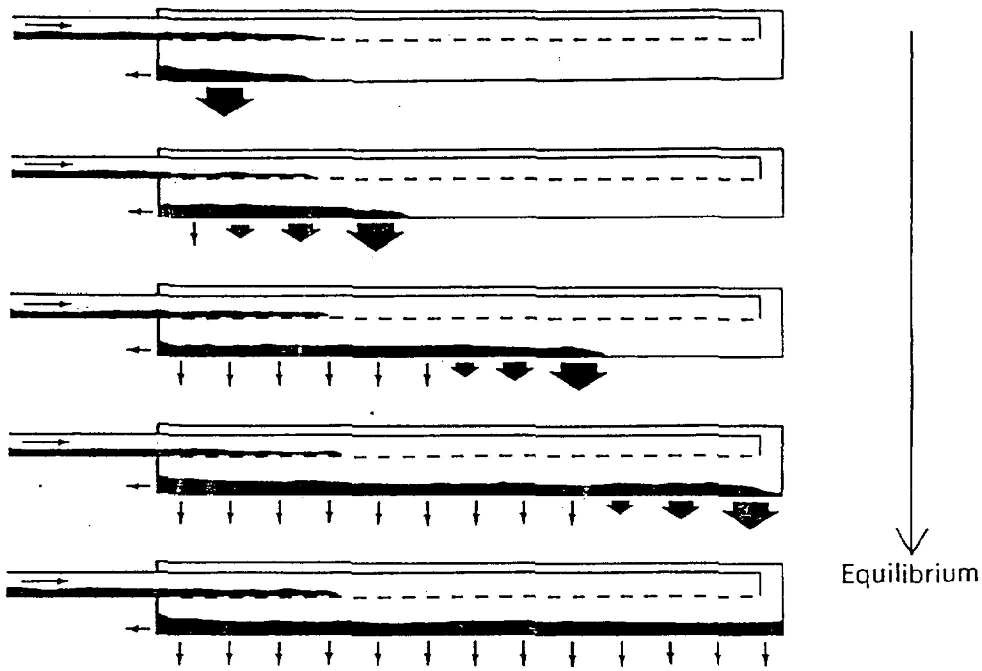


Figure 17. Progressive clogging of the infiltrative surfaces of subsurface absorption systems (11)

Repair Strategies
* Loading Rates *

- Hydraulic loading rates (LTARS)
- Unaccounted for sources
- Dosing
- Distribution
- Water use reduction
  - passive devices
  - active measures (behavior changes)
- Water use management
- Surface water diversion

Repair Strategies
* Physical Causes *

- Replace damaged components
- Tree roots
- Improper venting

Repair Strategies

* Landscape Position *

- Relocate drainfield
- French drain
  - limited solution
  - critical design
- Management of surface run-off
- Water use reduction

Repair Strategies

* Soil Related Problems *

- Water table
  - shallow installation
  - pretreatment

- Slow rates
  - shallow installation
  - increase size
  - improve distribution

Repair Strategies

* Soil Related Problems *

- Soil clogging
  - evaluate longevity of previous system
  - replace drainfield
  - evaluate size and condition of tank
- Plastic clays
  - water use management
- Restrictive horizons
  - shallow or deep installations
  - french drain
  - modify size
  - improve distribution
  - modify trench spacing

Component Functions
* Pretreatment *

- Septic Tank vs Dual Septic Tank
  - tees
  - Zabel filter
- Sand Filters
  - trickling
  - intermittent
  - recirculating
- Aerobic Pretreatment
  - Class I
  - Class II
- Constructed Wetlands

Component Functions

* Final Treatment and Disposal *

- Drainfield
  - distribution boxes
  - flow diversion valves
- Enhanced flow
- Low pressure distribution
- Elevated sand mounds
- High pressure drip disposal

Component Functions
* Blackwater Treatment *

- Composting toilets
- Vault privies
- Incinerating toilets

The Systematic Evaluation and Repair of Failing Drainfields in the Coastal Zone Area of Virginia

By: Donald J. Alexander, M.S.
Calvin Jones
Paul Sandman, M.S

The Systematic Evaluation and Repair of Failing Drainfields in the Coastal Zone Area of Virginia

By: Donald J. Alexander, M.S.
Calvin Jones
Paul Sandman, M.S

Acknowledgements

This manual was funded in part, by, the Virginia Council on the Environment's Coastal Resources Management Program through Grant #NA17OZ0359-01 of the National Oceanic and Atmospheric Administrations, Office of Ocean and Coastal Resource Management, under the Coastal Zone Management Act of 1972 as amended.

First Printing September 25, 1992

The Systematic Evaluation
and Repair of
Failing Drainfields in
the Coastal Zone Area of
Virginia

Table of Contents

Chapter 4
Repair Strategies

Chapter 5

Component functions

Chapter 1

Introduction

Historically, drainfield repairs have not received the attention they deserve. Considered as messy and foul smelling work, drainfield repairs often have been held in low esteem. In some localities repairs have been relegated to the newest environmental health specialist. To be consistently effective, repairs must receive in depth evaluation and consideration before a repair permit is issued.

Identifying the cause of failure is an essential element which needs to occur before selecting a method of repair. This has not always occurred. The unfortunate result has been the permitting of new systems which will likely fail because the cause of the failure was never accounted for. In other cases, systems have been replaced that did not need to be replaced. A $2,500 repair system is installed (and will soon fail) when the real culprit is a leaky toilet valve that can be fixed by the homeowner for under $20.00.

The purpose of all onsite systems is to treat and dispose of effluent. Most systems are recognized to be failures when they no longer can dispose of effluent. Rarely have systems been considered as failing when they were polluting ground water. Most systems that are not disposing of effluent are also failing to treat it as well. In many instances, the failure to effectively treat the effluent has occurred long before the system ceases to dispose of effluent.

From a public health perspective, sewage treatment is at least as important as disposal. This manual is intended to provide the specialist with guidance on how to evaluate and repair failing drainfields. The manual consists of four major sections. The first section addresses the causes of failures to familiarize environmental health specialists with the scope of reasons why drainfields fail. The second section provides a systematic approach to identifying the cause of a specific failure. The third section addresses strategies to repair systems in a manner that will improve treatment and disposal under the Department's current mandate to protect public health and the environment. The fourth section addresses system components and looks at their application in repairing a failing system. In general, the manual emphasizes pretreatment and water use management.

Not every site has a perfect solution. This manual does not pretend to have a solution to every problem. However, after the limits of a site are recognized, essentially every site can be managed to give improved levels of effluent treatment and disposal. The expense and life style changes associated with some repair systems may not make the solutions attractive on a widespread basis. While these repairs are not acceptable for new construction, they may be preferable to vacating an existing dwelling.

The goals of this manual are as follows:

1. To protect and enhance public health protection by providing thorough and appropriate solutions to repair failing sewage systems.

2. To provide for cost effective, long lasting sewage system repairs.

3. To provide environmental health specialists with the knowledge to make competent repair decisions and to communicate this effectively to homeowners.

4. To meet the needs of our customers (citizens with failing drainfields) through the delivery of our services.

Treatment

Wastewater is comprised of many constituents. Some of these must be removed or rendered harmless before the water is disposed of to be used again. This is what treatment is all about.

Treatment is a relative term and often means different things to different people. What may be considered as sufficient renovation for one person may be entirely inadequate for another person. For example, sewage entering the septic tank is 99% water in most residential sewage. While it is nearly "pure" water, it certainly isn't safe. Effluent from the septic tank can be treated by several means to reduce bacteria levels by another 99%. While this sounds like a high level of treatment, it does not mean the water is anywhere approaching drinking or swimming water quality. Effluent leaving the septic tank typically contains 10^8 or more fecal organisms per 100 ml. A 99% reduction is equal to a two log reduction and results in 10^6 fecal organisms per 100 ml. In more familiar terms, this water contains between 10 million and 100 million, fecal organisms per quart. This is hardly "pure" and certainly is not an acceptable level of treatment for human consumption according to any public health standard.

Biological Contaminants

Bacteria

Domestic sewage contains a variety of organisms including bacteria, viruses, and parasites. Some of these organisms, but by no means all, can cause disease. These disease causing organisms are called pathogens. The presence of these harmful organisms is measured indirectly by testing for fecal organisms. Fecal organisms represent a health hazard and indicate the probable presence of disease causing organisms. Wastewater treatment should be designed to remove this potential.

Fecal coliform and fecal streptococcus bacteria are commonly found in wastewater. These bacteria live in the intestinal tracts of healthy humans (and other warm blooded animals). Both bacteria are used as indicator organisms to establish whether a source of water is contaminated by sewage. The ratio of fecal streptococcus to fecal coliform can be used to indicate whether the source of contamination is human or animal. The test however lacks sensitivity. More often than not the results of the ratio are ambiguous and unable to differentiate between sources. Usually, the fact that water is contaminated is more important than the source.

In terms of treatment, residential wastewater usually contains 10^8 to 10^{10} organisms per 100 ml. The septic tank provides little or no improvement in the biological quality of the effluent. Biological treatment of effluent occurs in the aerated soil beneath and around the drainfield. Harmful bacteria from the wastewater must compete with native soil bacteria and organisms in the biomat around the trench. Those organisms that pass through the biomat at the gravel-soil interface must be physically filtered by the soil or die off due to time and hostile environmental conditions in order to render the wastewater harmless. From a public health perspective, this must occur before the wastewater is used again by humans.

Most pathogenic organisms have a narrow range of environmental conditions under which they can survive. The warm, wet, anaerobic environment of the intestines is ideal for the survival of these organisms. The colder, dryer, aerobic environment of a well drained soil is usually unsuitable for the reproductive and survival needs of these organisms.

Viruses

Viruses are much smaller than bacteria and soil particles. They are not physically filtered by soil. They are however, retained by the soil using a very different mechanism. Soils have a property known as cation exchange capacity (CEC). This refers to the soil's negative electrical charge and ability to hold positively charge particles. The positively charged viruses then attach to negatively charged soil particles as they move through the soil. This process is called adsorption. Cation exchange capacity is dependent upon the chemical make-up of the soil and the available surface area of the soil. That is to say, the CEC of a clayey soil will almost always be higher than that of a sandy soil.

Viruses consist of a nucleic acid (either RNA or DNA) and a protein coating. The protein coating is positively charged and because of the organisms extremely small size, it will be adsorbed to the negatively charged particles in a soil. As a result, it should be evident that a significant clay fraction (even that found in texture group II soils) is necessary for optimum effluent treatment. Sandy soils can treat bacteria well, while allowing viruses to pass essentially unaltered.

Physical and Chemical Contaminants

Biochemical Oxygen Demand

One of the measures commonly employed when evaluating wastewater is the five day biochemical oxygen démand test or BOD_5. This test is used to measure the strength and biological stability of wastewater. It does this by measuring how much oxygen must be consumed to biologically digest and chemically stabilize the organic components of the wastewater. Typical residential wastewater has a BOD_5 between 200 and 250. In and of itself, BOD_5 is of no public health significance. It does however, provide a tool for defining the characteristics of a particular waste flow. This can then be used to establish appropriate treatment methods.

Suspended Solids

Suspended solids in septic tank waste typically are the result of material close to the specific gravity of water. Consequently they do not settle out in the less than perfect settling basin called the septic tank. Suspended solids include filamentous material, hair and biological wastes.

It should be noted that suspended solids in the septic effluent do not contain much BOD. Most of the BOD is dissolved in the liquid portion of the effluent. The major problem with high suspended solids in effluent is the physical clogging of pumps, LPD orifices, and soil pores in the absorption field.

Nitrates

Nitrogen is generally considered to be the single most important chemical constituent in domestic wastewater. From a public health aspect excess nitrates can cause methemoglobinemia. Environmentally they can cause eutrophication in streams and rivers and are also the limiting nutrient in salt water estuaries.

Nitrates in domestic wastewater are produced almost entirely as a result of the conversion of the nitrogenous wastes in urine to ammonium and then to nitrate. In the presence of a carbon source in an anaerobic environment, the nitrate can be reduced to nitrogen gas and carbon dioxide. The typical situation has the nitrogenous wastes being converted to ammonium in the septic tank and then to nitrate in the aerobic environment of the drainfield. Nitrates are highly soluble in water and do not bind to soil. As a result, nitrates are highly mobile.

At levels in excess of 10 mg/l nitrates may cause "blue baby syndrome" (methemoglobinemia) in infants. Infant hemoglobin has a higher affinity for nitrate than for oxygen. Infant hemoglobin is replaced by adult hemoglobin as the child grows. By the age of six months, the replacement is sufficiently complete that the occurrence of "blue baby syndrome" is unlikely or impossible. The result of this illness is that infants essentially suffocate because their blood can no longer carry oxygen (Baum, 1982).

While this is a very serious, potentially fatal, condition, it is by no means common. Reasons for this are that nitrate levels from septic systems generally will not exceed 10 mg/l in the ground water except where unusually high development densities occur. The Virginia Department of health requires the designers of all systems with design loading rates in excess of 1,200 gallons per acre to address nitrate contamination in the ground water. Even if the 10 mg/l limit is exceeded, there is no guarantee a case of blue baby would occur. Prior to the illness occurring, a drinking water well must intercept the contaminant plume from the drainfield and a susceptible infant under the age of 6 months must consume quantities of the water sufficient to cause the illness.

It is important to note that onsite wastewater systems are but one of several significant contributors of nitrates to ground water. Agricultural fertilizers, animal wastes (especially when concentrated in areas such as feed lots), and residential lawn fertilization all can be sources of nitrogen. Of the potential sources of nitrogen, residential onsite systems are generally the least significant contributor, but nonetheless should be considered.

Chapter 2

Causes of Failure

Before a strategy to correct a malfunction can be developed the problem cause must be identified. The causes for the failure can be varied and might be attributed to a number of different factors. Four primary reasons exist for system failure. These are: hydraulic overloading, physical component failure due to stress or deterioration, landscape position which contribute excess water to the absorption field and unsuitable soil conditions.

Causes of Hydraulic Failure

Hydraulic failure occurs when more effluent is applied to an absorption field than can be disposed of by the field. Hydraulic failures express themselves as continuous or seasonal wetness over the drainfield. There are several causes of hydraulic failure but all are related to either excessive loading rates, organic clogging or a combination of the two. In addition to the obvious health and nuisance problems associated with surfacing effluent, some limited and inadequate wastewater disposal may also be associated with this type of failure.

Loading rates

The hydraulic conductivity of a soil will dictate the acceptable hydraulic loading rate for a given site. Long term acceptance rates (LTAR) are normally significantly less than measured short term rates. Loading rates are generally construed to mean the number of gallons of effluent applied per square foot of absorption area. In this manual, the loading rate is the number of gallons of effluent and water from all sources applied per square foot of absorption area. One of the key strategies to repair many systems is to eliminate all extraneous water and thereby prevent it from being applied to the absorption field.

Excessive loading rates are probably the leading cause of drainfield failure. This type of failure may be caused by excessive water use by the individuals occupying the house or an excessive number of occupants in a house. The end result is, water use exceeds the design capacity of the sewage disposal system.

The causes of excess water use are as varied as the residents of the Commonwealth. In some upscale neighborhoods hot tubs and Jacuzzi's have contributed to hydraulic failures. Informal conversations with field staff and consultants indicate that this may be a growing problem. Additionally, other uses such as a small scale commercial laundry, a beauty shop, or even hobby related uses such as darkroom processing can lead to premature system failure. Even illness can affect water use. One of the authors has observed a failure due to waste from a kidney dialysis machine over-loading an older, undersized drainfield. During the investigation of the failure the homeowners should be interviewed to determine what water use patterns are occurring and if any additional loads are being place on the system for which it was not designed.

Leaking fixtures

One prime cause of hydraulic failures is leaking plumbing fixtures. Whether it is a sink or bathtub with a steady drip or a toilet that never stops running, awareness of the problem and its implications is low. These leaks are often neglected until the problem surfaces in the yard. A small leaking facet or toilet can easily add the equivalent of an extra bedroom to the daily waste flow. More serious leaks, or multiple leaks, can double the estimated the estimated wastewater flow.

Uneven distribution

Uneven distribution of effluent in the drainfield may result in premature failure of the soil absorption system. Conventional sewage disposal systems use a distribution box to equalize the flow to the drainfield. Under near perfect conditions, equal distribution is difficult to achieve using a distribution box. The resulting uneven flows may or may not be significant enough to cause any problems with systems installed on flat terrains. On the other hand systems installed on sloping terrains may develop problems should the downhill lines receive more water than the other lines of the system. The problem may be compounded if care is not taken to prevent the box from being disturbed during the back-filling process.

Physical Causes of Failure

Tree roots

When the drainfield is located in or near a wooded area, tree roots may cause problems. Trees with shallow root systems will seek water from the drainfield lines. A network of roots can enter in the drainfield and grow back to the distribution box and septic tank blocking the flow into the drainfield. Maple trees, alders and other water loving species should be considered as possible sources of roots entering a system even when located 100 or more feet away from the problem area.

Age, physical disturbance and material failure

Some components will outlast the useful life of the system while other components deteriorate over a shorter period of time. Components such as Orangeburg pipe, cast iron septic tank tees and some concrete products are system elements that tend to fail early. Plastic pipe, plastic distribution boxes and some concrete may never deteriorate (significantly) with age but, like any product, can be damaged when stressed. Driving over system components, plowing or tilling over a system and other disturbances such as utility installations can damage components.

Clogging or organic mats

Clogging mats are another cause of failure and are associated with older systems and systems that have received poor maintenance or abuse. An organic or zoogleal mat will form on the soil gravel interface of all properly working systems. Without any other external factors, over time, even the best soils cease to absorb effluent quickly enough to keep up with wastewater production.

Use and maintenance can either retard or accelerate clogging. One of the prime accelerators of mat formation is the garbage disposal. Use of a garbage disposal increases the organic loading rate placed on the system. Compounding the problem is that a significant portion of the solids are composed of cellulose, which is resistant to biological breakdown in the septic tank. Additionally, any water use which scours the tank, such as discharges from hot tubs or Jacuzzi's, will increase the organic loading rate placed on the drainfield.

Landscape Position Related Problems

Infiltration

All onsite sewage disposal systems are susceptible to ground water infiltration if the septic tank and pump chamber have not been properly sealed. This occurs most frequently when the house is built low on a slope. The resulting position of the septic tank and pump chamber is such that they are likely to be placed in an area with a high seasonal water table. When the pump discharges, emptying the pump chamber, surface water can flow directly into a leaky pump chamber or enter from leaks in the septic tank. In essence, the pump chamber and septic tank serve as a means of dewatering the area where they are installed. This will result in hydraulic loading rates on the drainfield greatly in excess of the design rate.

The placement of the septic tank and pump chamber in a drainway may also result in the sewage disposal that disposes of surface water which has been directed toward the tank or pump chamber. Inadequate or improper surface drainage may be another contributing factor to hydraulic overloading. Careful attention should be paid to surface drainage, footing drains and roof run-off in the vicinity of the drainfield. The removal of surface water and the diversion of footing drains and roof run-off will lessen the amount of water which must be disposed into the drainfield.

Soil Related Causes of Failure

High water table

High seasonal water tables are another cause of failure for a sewage disposal system. High water tables usually occur because naturally occurring site and soil conditions cannot dispose of precipitation falling or flowing onto the site. The wastewater from a home will compound the problem. The most significant problem with high water table soils is the greatly reduced treatment efficiency of the soil. Renovation of effluent in saturated soils requires much greater time and distance than treatment in unsaturated soils (Reneau and Pettry, 1975).

Soils having drainfields installed in a seasonal water table will also have a difficult time properly disposing of both the naturally occurring precipitation and the added wastewater. These systems will experience anaerobic conditions in the drainfield resulting in the formation of a biological mat and soil reduction processes sooner than systems installed in well drained soils. In areas of high water tables the life of a sewage disposal system will usually be much shorter than in areas with well-drained soils.

Slow infiltration rates

Soils with slow infiltration rates may exhibit yellow and red mottles, pale brown coatings on soil ped faces and root channels and may occasionally show grey mottling. These observed soil characteristics indicate that the soil is having difficulty transmitting the amount of precipitation infiltrating in the immediate area. Typically in Virginia this is about 40 inches per year. Adding a drainfield to the site will typically add an equivalent of 120 inches or more of rainfall per year.

Treatment and disposal in these soils varies with seasonal rainfall. In the spring and fall when rainfall is high, treatment will deteriorate. This occurs after periods of prolonged saturation, which routinely happens late in the winter and through the spring, until leaf out occurs. When the water table is high, both wastewater treatment and disposal become serious problems. These soils are responsible for most seasonal failures.

Plastic clays

Plastic clays or high shrink-swell soils generally do not provide any acceptable level of treatment and disposal. These soils contain active clays, such as montmorillonite, which expand when wet and shrink when dry. When a drainfield is installed in shrink swell soils, effluent causes the clays to swell shut and failure is inevitable.

Restrictive horizons

Restrictive horizons impede the downward movement of water. They may also be the cause of "perched" water tables. Their impact on the operation of a drainfield depends on several factors. Their proximity to the trench bottom, their degree of permeability, and their continuity all contribute to their relative importance. The closer a restrictive layer is to a drainfield and the less permeable it is the greater the adverse impact of the restrictive horizon. Additionally, some types of restrictions may not be continuous across a drainfield site. Fragipans in some parts of the state are extremely discontinuous. The practical result of a discontinuous restriction is to make the functional portion of the drainfield too small unless the restriction is accounted for in the system design.

Chapter 3

Systematic Approach to Evaluating System Failures

Introduction

Septic tanks and drainfields have been described as a temporary means of sewage disposal. Some systems last for many years when installed under favorable conditions; however, even those systems will eventually fail. Other systems may experience premature failures due to hydraulic loading, soil conditions, or a number of other external factors.

No matter what the reason for a system failure, the health department will be involved with correcting the problem. There will be far greater pressure on the environmental health specialist to provide a correct solution than if he or she were evaluating a vacant lot. There may be poor soil conditions, inadequate space, and a number of other factors making a proper solution far more difficult. The property owner with a considerable amount of money invested will expect the specialist to provide answers to correct the sewage disposal problems. The health department may not have the answers to all the problems, but we owe it to the homeowner to provide the best advice possible. This can be done this by using a systematic approach in evaluating the problem.

The key to correcting any problem is the proper diagnosis of the cause of the problem. In the case of a leaking toilet, adding drain lines to cure the saturated drainfield, will provide very short term relief. If the cause of the problem is not addressed, the problem will reoccur. Be careful not to fall into the trap of assuming the cause of drainfield failure before thoroughly investigating the situation. Failure to use a systematic approach could result in an embarrassing situation for the specialist and unnecessary expense for the homeowner. Take your time and use good common sense when evaluating an onsite sewage disposal system problem.

In making a systematic evaluation of the problem there will be three major component areas to consider. The first part of the evaluation will consist of the house, plumbing, and hydraulic loading. The second major component area will be the septic tank and appurtenances (tees, sewer line and effluent line). The third and final component area will be the distribution box and drainfield. When making an evaluation it is important to start with the house and plumbing and work toward the drainfield. Following this approach will assist the specialist in identifying the source of the problem and providing the best lead in finding an appropriate solution.

The House and Plumbing

Starting in the house, survey the occupants of the house and make an estimate of the amount of water being generated by the household. If the house is connected to a public water supply, actual water use figures should be obtained. In a rural setting this will probably not be possible. However, in such a case, the flow can be estimated by asking a few simple questions. The first thing to address is the number of occupants in the house. This will yield a fair indication of what the base household water use is in gallons per day (GPD). Next, it is necessary to determine if there are any other unusual uses that would modify the base estimate. They may be operating a business or have a hobby which would generate additional water use. Instances have occurred where day care centers and small scale laundry services have been operated out of residential dwellings. Determine what type of fixtures and facilities are in the house. Fixtures such as hot tubs, jacuzzi's, and garbage disposals are all items which contribute to increased water flows and in some cases increase the organic load placed on the drainfield. Determine if any type business is being operated out of the home such as a beauty shop, or maybe they have a darkroom. By questioning the occupants regarding the type of uses placed on the system, a reasonable assessment of water use can be made.

<u>Sewer line blockages</u>

While inside the house, evaluate the extent of any plumbing back-ups. Determine if the problem is occurring with all fixtures in the house or a single fixture. There have been cases where people have called the health department because one toilet will not flush while all other fixtures in the house are working satisfactorily. This usually indicates a blockage in the plumbing which can be repaired with comparative ease. On occasion, toys, diapers or other items have been known to completely block the sewer line causing all fixtures in the house to back up.

In some instances the kitchen drain may have become stopped-up due to grease. Occasionally the kitchen and laundry drains have been connected to a grease trap which is separate from the rest of the septic tank system. If the problem can be isolated to one fixture, or a common group of fixtures, the homeowner may need a plumber and not a septic tank contractor. Assure that the sewer line is open before proceeding further with the evaluation.

Leaking fixtures

Another essential element that the specialist must check for is leaking toilets or fixtures. The leaking toilet is one of the most common and overlooked causes of unintended water waste. Be sure to ask the homeowner if they have any toilets which "hang-up" or "run-on" after being flushed, or in some way do not do not operate properly. Because homeowners are not always aware of these problems, the specialist should ask to re-check all the toilet fixtures. One way of checking a toilet for leaks is to put a drop of food coloring or vegetable dye in the toilet tank and wait ten minutes to see if it leaks into the bowl. If the coloring leaks into the bowl, the plunger ball in the toilet is the most probable cause of the leak. The homeowner needs to check the plunger ball to see if it is obstructed with grit or debris at the seal or if the seal is worn and needs to be replaced.

Ask the homeowner about leaking faucets or faucets that are difficult to shut-off. Ask permission to double check these. This can be conveniently done while checking for leaking toilets. A plumber should replace the washers on dripping faucets. Either a leaking toilet or faucet can increase wastewater flows sufficient to cause system failure. Be sure to thoroughly investigate the home for leaks before proceeding further with the evaluation.

Septic Tank

Maintenance

Once the hydraulic loading rate has been established and the house fixtures and plumbing eliminated as a problem, determine how well the system has been maintained. When was the last time the septic tank was pumped and how often has the tank been routinely pumped. If the tank has to be pumped frequently during the winter months, the drainfield may be installed in soils with a seasonal water table or in slowly permeable soils.

Records and history

Obtain as much history as possible on the system. Often, the past performance of the system can shed some light on current problems. Did the system malfunction gradually or did it occur suddenly? Be very suspicious of failures that occur suddenly without warning. These types of failures usually occur when a component fails or the water use increase sharply. A gradual failure is more likely an indicator of system failure in the drainfield. This type of failure may start out as a small wet area in the yard that gradually increases in size. If drain lines are added to a sudden failure, without correcting other problems, the problem may reoccur. The environmental heath specialist should review the health department records of the system design and discuss the history of previous problems with the homeowner. Department records should indicate the design capacity of the system, general design, system location, and the age of the system.

When records on a system cannot be found there are several other visual and mechanical ways to locate the system components. The septic tank is normally (but by no means always) located near the house where the main sewer line exits the house. This can be located by looking in the crawl space or basement. When access inside the house is limited, locating the vent pipes on the roof can help locate general plumbing locations. It then becomes a matter of second guessing what a plumber might have, should have or could have done to get the plumbing outside. The septic tank is usually located in the shortest line between the house and the drainfield. The drainfield can often be identified by the deeper green color over the trenches and sometimes by settling over the trenches. Additionally, look for unusual lawn growth to assist in locating the tank. A leaking tank may give rise to lush growth, while a tank with minimal cover may not support grass well at all. With some regularity, the exact shape of the tank is revealed in the lawn.

When all else fails, don't hesitate to have the homeowner hire a plumber or septic tank pumper locate the system components. Experienced professional are equipped to snake out component locations and are more adept at probing and locating the parts of a septic system than most environmental health specialists. Often times they have been to the site several times before the health department is called for assistance. Use these individuals as a resource and learn what has been done to repair the system.

After the components are located, the environmental health specialist should request the owner to have the septic tank and distribution box uncovered to evaluate these key components. The tees in the septic tank are essential elements of the system and cannot be checked any other way. Older systems were usually installed using cast iron tees. Cast iron tees will corrode and stop up. This can happen at either the inlet or outlet tee. Septic tank pumpers have been known to removed these tees to snake out the conveyance line. All too frequently they do not get replaced. One should therefore note if the tank still has both tees.

If the tank has been pumped recently, it is often wise to solicit information from the septage hauler. Often they can offer observations about the system, such as water flowing back into the tank or other unusual conditions, that can aid in identifying the problem. With the tank uncovered the specialist can observe the liquid level in the tank. If the liquid level is at normal flow level, but the owner cannot use the fixtures in the house, check the inlet side of the tank. This condition indicates a blockage in the sewer line or the inlet tee. If the septic tank is overflowing at one of the access lids the outlet tee should be checked. The most probable cause of this condition is a clogged outlet tee or effluent line between the tank and the distribution box. If the outlet tee is clogged, the drainfield may not need to any repair.

The conveyance line from the septic tank to the distribution box is another area where problems may occur. Many older systems used Orangeburg pipe for the conveyance line when they were installed. Orangeburg pipe is made of tar and paper rolled into a pipe. This type of pipe can blister on the inside and close the line off. It is no longer used today and should be replaced whenever a system is repaired. Regardless of the material used, the conveyance line may have been damaged when the system was backfilled, when the yard was landscaped or at some later date. Be sure to look for evidence of traffic use over the system. This is especially important in the spring when the ground is wet and cannot bear as much weight. This is when traffic damage is most likely to occur.

Drainfield and Distribution Box(es)

The next step is to uncover and examine the distribution box. If the water is at normal flow level (even with the bottom of the outlets), the problem should be back towards the house or the hydraulic failure is occurring at a lower elevation. On the other hand, there are several possibilities if the outlet ports are covered with effluent. Some potential problems are drainfield failure due to clogging, a high seasonal water table, poor soil permeability, tree roots (or other blockage) in the header lines, or other problems.

Investigating the drainfield area is next step in the process. This should include a soil evaluation performed in the drainfield area. If all else is working properly, it is assumed that the drainfield will be saturated. A boring or two should confirm this in short order. If the drainfield is not saturated, the problem is either in the header lines or has been missed and lies somewhere back toward the house. If the drainfield is saturated, the next question to be answered is why?

Surface drainage and other water sources

The saturation may be caused by improper area drainage. Included in the drainage evaluation would be roof and footing drains which may be directed on the drainfield area. Excess drainage from swimming pools, hot tubs and water softeners may be adding excess water to or over the system. Surface water management is equally important. Does the area have positive surface drainage or does water collect over the drainfield? Are there paved areas generating a large amount of surface run-off onto the site? Are there electric, gas, cable TV or other underground utilities which may divert off site water into the drainfield? Is there a lawn irrigation system is used excessively? Are there any old water lines that may cross the drainfield area? Older water lines were often galvanized and tend to develop pin hole leaks which could flood a drainfield. Has the owner planted trees in the drainfield area?

If the sewage disposal system has a pump, a leaking septic tank or pump chamber must be considered. Typically you may find the house, septic tank, and pump chamber located in an area with a high seasonal water table and the drainfield in a remote area where soil conditions are better drained. If the septic tank and pump chamber have not been properly sealed ground water will leak into them. When this happens a hydraulic overload could be created by this additional water. The environmental health specialist should be aware this possibility exists when evaluating a drainfield problem.

Trees

Roots from trees have also caused septic system problems. Explain to the owner the effects of trees on the septic tank system. Encourage the owner to remove all trees with shallow root systems from the drainfield area. Water loving species such as maples, alders and willows should be removed from more than just the immediate area over and in close proximity (ten feet) of the drainfield. It is suggested that these trees be kept at least fifty feet away and further is better.

Soil conditions

Poor soil conditions as well as age can cause a drainfield to become ponded. Soil conditions which can cause failure have already been briefly discussed. It is important to note that even good soils will clog over time and cease to operate satisfactorily. These systems are typically twenty-five years old or older, and exhibit a dark gray or black mat at the edge of the drainfield-soil interface. Soil borings in the drainfield area and in the drainfield itself are essential to determine the cause of failure. As with any soil evaluation, it is essential to keep good notes on the observations made while boring.

Closing perspective

If you are lucky enough to have the homeowner(s) present, take time to educate them on how to properly care for a septic tank system. Let them know there is a difference between a city sewer system and a septic tank system. Provide pamphlets which outline the care and maintenance of the septic tank when the operation permit is issued. The more the owner knows about the septic tank system the greater the chances of it being maintained and working longer.

The most important aspects of making a good evaluation are using common sense, good judgement and following a systematic approach. However, simply knowing what caused a failure is only the first step in solving the problem. Once the problem has been properly identified, an appropriate solution to the problem must be developed to correct the problem.

Chapter 4

Repair Strategies

Introduction

This section of the manual is intended as a starting point for finding solutions to failing drainfields. Environmental health specialists looking for a panacea or a definitive answer to every problem will be disappointed. They don't exist. Almost every site can be improved upon. Many sites can be repaired to the satisfaction of all parties, while other sites can only be managed to reduce potential health risks. A careful review of the strategies in this section will help the specialist distinguish between the repairable and the manageable and to choose an appropriate course of action.

Once the problem with the sewage disposal system has been diagnosed a strategy for the correct repair must be developed. It is very important to properly identify the cause of the problem before making a recommendation to correct it. Strategies may differ based on the nature of the problem or combination of problems. The approach to correct a seasonal problem may be entirely different than the strategy used to correct a 30 year old system failing due to organic clogging. Both treatment and disposal of the effluent must be considered when developing a method of repair. If treatment can be achieved and is economically viable, it should be accomplished.

General Concepts

<u>Hydraulic loading rate</u>

Hydraulic infiltration decrease as systems age while loading rates tend be constant or increasing. Organic clogging, prolonged anaerobic conditions, high carbon-nitrogen ratios, high BODs and suspended solids all contribute to accelerated reductions in infiltration rates (Avnimlelech and Nevo, 1964 and Kristiansen, 1982). A reduction in the hydraulic loading rate or flow in some cases will provide a viable repair option. Dosing the system will enhance aerobic conditions within the system by providing alternating periods of wetting and drying. Reducing flow will also enhance the ability of the system to operate under aerobic conditions by reducing the amount of liquid to be disposed of. The advantages of dosing are negated when done in conjunction with systems installed in a water table because these systems will remain flooded. In order for dosing to be effective, an aerobic period is necessary.

Water use reduction and management

Water management is the key to drainfield longevity and assuring effective system repairs. Water use reduction may be broken down into two major classes: active and passive. Active water use reduction requires changes in lifestyle and water use habits. Passive water use reduction requires the installation of water saving fixtures and does not appreciably alter the users lifestyle.

Water use reduction can be achieved by installing water saving devices. These items can be purchased from many hardware stores, plumbing suppliers and home centers. Some may even be installed by the homeowner. The simplest of these would be water saving shower heads, aerators for faucets, and water displacement devices that can be placed in the toilet tank. The toilets on the market today typically use 3.5 gallons per flush and water saving toilets are available that use 1.6 gallons. Many new home appliances are designed to reduce water flows when used properly. The use of water saving appliances should be encouraged. In addition to the many mechanical devices available, there are conservation measures the homeowner can apply to everyday living habits to reduce water flows. Local utility departments usually have this type information in booklet form for the public's use.

Passive water saving devices

Generally these will be the most effective methods of achieving modest but consistent water use reductions. They include the use of water saving toilets, water saving shower heads and in line flow restricting devices, low flow faucets and low water use appliances. Typical toilets today use 3.5 gallons of water. Effective toilets that use as little as 1.6 gallons per flush are available. Flow from shower heads can be reduced effectively to three gallons per minute without sacrificing the quality of the shower. When possible, flow restrictors should be used rather than low flow shower heads. Shower heads are readily replaceable and are not as permanent as in line flow restrictors.

Active water saving measures

These are measures that require lifestyle changes on the part of the homeowner. Their effectiveness will be highly variable depending upon the cooperation of the occupants. As soon as there is a change in ownership in a dwelling that's restricting its water usage, there is a high probability that water use reductions will not be continued. These methods are included as educational and informational material. Wherever possible, rely on passive water saving devices. Active water saving methods include the following actions:

1. Exclusive or seasonal use of a laundromat for clothes washing.

2. Modifying when laundry is done at home to avoid peak periods of water use. Laundry is done throughout the week as opposed to on a predetermined laundry day.

3. Only full loads of clothes are washed.

4. Showers are closely timed or the shower is not run continuously while washing (i.e., soak, lather with the water off, then rinse).

5. The dishwasher is not run daily and only run when there are full loads.

6. Toilets are not flushed after every use.

Compliance with these suggestions is impossible to measure or enforce and the Department is not suggesting that staff attempt to do so. These suggestions are included so that homeowners with especially onerous failures can be advised how they might manage further water use reductions.

Shallow placement

Prior to 1982 the emphasis for sewage disposal system design was placed upon the disposal of effluent rather than treatment. Today's strategy is to repair these systems in ways that maximize treatment without sacrificing disposal.

Shallow placement offers the advantages of greater separation distances to the seasonal water table, a greater ability to operate under aerobic conditions, and generally better soil textures.

Achieving shallow placement may be as simple as raising the plumbing under the house and where it exits the house or may require the use of a pump chamber and sewage pump. Each of these methods of achieving shallow placement has its own advantages. Raising the plumbing may have more initial economic appeal and avoids all concerns of a mechanical failure of a pump. If a pump chamber and sewage pump is used, the system can be designed for enhanced flow. This type of system provides dosing and resting cycles which improves aerobic treatment.

Another strategy which could be employed is the use of an ultra shallow placed low pressure distribution system. This system would offer the advantage of equal distribution over the entire drainfield area. The cost of this system will be slightly higher but it offers a greater longevity. In areas where the seasonal water table is a major concern, a modified sand mound may provide the best solution.

<u>Grey water separation</u>

Grey water separation can reduce the hydraulic loading on a drainfield. Typical water usage from a clothes washer varies from 30-60 gallons per full cycle. A household with small children will add many gallons of wastewater to be disposed of in the septic tank system. Under the present <u>Sewage Handling and Disposal Regulations</u>, it is estimated laundry wastewater accounts for 20% of the total wastewater volume. If the grey water were placed on a separate system or eliminated, a minimum 20% flow reduction would be expected on the existing drainfield system. Other sources place this figure even higher. This reduction in water flow could possibly be enough to allow the system to function properly.

<u>General comments on failing systems</u>

There are few general comments that can be made about failing drainfields. Each is a unique situation calling for individual thought and evaluation. Two general concepts which do seem to have broad application are the use of water saving fixtures and the need to correct problems expeditiously. There is no cause of failure that will not benefit by reducing water flow to the system. Septic tank efficiency improves and the amount of effluent on top of the ground is immediately reduced. Drainfield failures that are allowed to continue indefinitely are a clear risk to public health. Additionally, in many cases failures that are caught quickly can be fixed easier and at less expense to the homeowner than if they are discovered years later. Drainfields that are allowed to remain continuously ponded undergo soil reduction processes which reduces the ability of the soil to absorb effluent. The sooner a site is evaluated, and a repair strategy is identified, the more likely portions of the system can be returned to service. While timeliness is important, there is no need to rush an evaluation: be thorough, be precise and use good judgement.

Addressing Hydraulic Overloading

The initial step is to identify and remedy the reason that caused the overload to occur. Damage to an overloaded system can range from minimal to total drainfield destruction. It is important that the specialist judge the extent of the problem and determine how much repair or replacement is necessary.

By boring into several of the drainfield trenches in several places along their length, the specialist can determine how much soil reduction (formation of gray mottles) and mat formation (black organic deposits) has occurred within the system. If there is little or no soil reduction, chances are good that the drainfield can recover from the failure without additional repairs. Occasionally it is necessary to block flow to one or more lines temporarily (three to six months) to allow them to recover. When this is attempted, it is essential that the specialist determine that the remaining portion of the system is adequate in both capacity and condition to handle the temporary increase in flow. Additionally, follow-up is equally essential to assure that the rested lines are placed back in service on time to prevent damage to the remainder of the system. A follow-up visit should be made within six months.

Systems with significant organic clogging (which should be expected in older systems) that also have significant soil reduction in the trenches should be abandoned. They generally cannot be expected to recover from the failure sufficiently to be useful in a reasonable amount of time (note: in a number years the laterals may "rejuvenate" themselves and be useful again. How long this process will take is the subject of some debate In general, severely damaged trenches probably need years, not months, in an aerobic environment to recover). Depending upon the severity of the problem these systems need to be either expanded or replaced. Decisions such as this need to be based on judgement and local experience. There is no substitute for experience.

<u>Uneven distribution</u>

The most common distribution problem is an out of level distribution box. Frequently, for a variety of reasons, the lowest lateral receives an excessive amount of effluent. Before determining the best method of repair the specialist must determine the extent of damage which has occurred. The techniques for determining this are given in the previous section on hydraulic overloading. In all cases the distribution method must be repaired. In some cases this alone will solve the problem. In other cases laterals may need to be rested or replaced. In extreme cases the entire system may need to be replaced.

In marginal soils (rates slower than 60 mpi or soils having a perc rate slower than 30 mpi and a water table within 18 inches of the trench bottom) replacement of the distribution box with a pressure manifold should be considered. This is initially expensive but can be cost effective if, in the long run the drainfield life will be increased. The dosing cycle which alternates aerobic and anaerobic conditions has been suggested to be excellent for prolonging absorption field life expectancy in certain soils. In very good soils and unsuitable soils, the difference in life expectancy may not be justified. A retrofit such as this includes adding a pump chamber and replacing the tight line to the manifold which is located approximately where the distribution box was located.

At sites where the expense of a pressure manifold does not appear warranted or feasible, enhanced flow, installation of a flow diversion valve, of even at grade access to the distribution box are alternatives that should be considered. A properly designed enhanced flow system will offer most of the advantages of a manifold distribution system. A flow diversion valve can provide long term alternating and wetting cycles which can benefit the life expectancy of the absorption field. An at grade distribution box, while the least acceptable alternative, can provide a practical means of removing individual lines from service as necessary.

Leaking fixtures

This is one of the most common causes or contributors to drainfield failure. Repair or replacement of the offending fixture or fixtures is the first step in correcting the problem. Homeowner education is essential because it is the only way to assure that the problem does not reoccur. Fixtures that fail once and are repaired will fail again in time. Therefore, take time to explain to the homeowner what to look for and how to test for leaking toilets. Discuss the cost of repairing a drainfield versus the cost repairing fixtures occasionally. Don't rush the discussion; take your time to make your point clearly and in a friendly manner.

Surface water and miscellaneous other sources

Surface water includes run-off from roofs, footer drains, basement sump pumps, driveways, patios, undeveloped upslope areas. Other miscellaneous sources may include discharges from water softeners, swimming pools, hot tubs, lawn irrigation systems and other uses. None of this water should be directed into or on top of a drainfield. Where problems such as this are encountered, the water should be diverted to another area or piped beyond the drainfield area before being discharged.

The improper management of surface water and other extraneous water is a major leading cause of failure. The importance of controlling surface water should not be underestimated when evaluating a failure. Permits for repairs should incorporate explicit information on controlling surface water.

Addressing physical causes of failure

Broken, blocked or damaged components

Repair in this instance normally requires replacing broken or damaged parts or removing any blockage in the system. It is important to determine if the situation is an isolated or a reoccurring problem. Occasionally changes in landscaping (such as extensive regrading or relocation of a driveway) can create a problem that needs to be corrected to prevent the problem from reoccurring. This is very different from damage occurring from isolated or nonroutine events which will not cause continuing difficulties.

Tree roots

The obvious solution is to remove the offending tree roots and reseal the points of entry to prevent infiltration, exfiltration and the re-entry of new roots. Removal of the offending tree is normally, if not always, necessary. There is anecdotal evidence that treatment of the system with copper sulfate will at least temporarily provide relief from root blockages. At this time there is no practical data indicating what the long term effects of such treatment are on groundwater or nearby vegetation. Further, proper dosages have not been documented to achieve the desired results without adverse impacts. Consequently, at this time the Department does not recommend the use of copper sulfate (or other compounds) for controlling unwanted root penetration into onsite systems.

Improper venting

Venting is done to allow air to freely enter the plumbing system. When a plumbing system is not properly vented, pressures less than that of the atmosphere may occur within a pipe or pipes. This can result in sluggish drains or complete backups. Improper venting is a problem that is rarely encountered in new construction due to adequate plumbing code provisions. Occasionally, older systems (>30 years typically) will have a venting problem. These problems are generally typified by sluggish plumbing and apparent intermittent blockages. Normally the drains are the first suspect. When no problem is discovered in the drainage system the vent system should be inspected and/or snaked out. In either new or old systems, a vent can be blocked by a birds nest or a dead animal (squirrels seeking innovative housing are prime offenders). In addition, vents branching off of the horizontal part of the sewer drain may fail to function properly in areas with a high seasonal water table. The vent should be located off of the vertical portion of the sewer line.

Addressing problems with landscape position

Systems in a concave position

When a drainfield is has been installed in a poor landscape position, little can be done to correct the problem short of relocating the system. Relocating the system should be the prime objective whenever this situation is encountered. When sites on the owners property are limited, easements should be considered.

When relocating a system is not possible, the only remaining option is to manage the failure to minimize the amount of effluent surfacing. A french drain uphill, and if appropriate on one or both sides of the system should be installed. A french drain is nothing more than a gravel filled trench designed to intercept water and pipe it to another location. Typically, they are constructed like a drainfield (occasionally with more gravel) and have the perforated pipe located at the bottom of the trench rather than the top. The preferred material is not three hole drainfield pipe, but rather slotted pipe like that used in footer drains. If drainfield pipe is used, placement of the holes either up or down is not significant (aside from lunch time debates).

The french drain should be anchored to a restrictive layer if one exists (i.e., installed so that the french drain trench bottom rests on or in the restriction). If no restriction exists, the drain should be installed at the same depth as the drainfield. The french drain should be installed 20 feet away from the absorption field if possible. Under no circumstance should the drain be installed any closer than ten feet to the drainfield. The french drain is not a cure all solution. It will help in most problem landscape positions but it will not cure the problem and should not be presented as such.

The area over the french drain should be graded to divert surface run-off into the drain and away from the drainfield. When possible, the drainfield area should be capped and crowned to improve run-off and reduce infiltration of any precipitation falling on the site. Surface water should be managed as described below. Water saving fixtures should be installed in the dwelling and the homeowner instructed on water conservation measures.

Addressing soil related problems

<u>Water table</u>

In this instance the reference to a water table means the presence of a water table at a distance less than the minimum required stand-off established by regulation. The primary difficulty in this situation in the coastal zone area is inadequate treatment capacity within the soil. Wastewater disposal is a problem but is frequently secondary in nature. Solutions to shallow depths to a water table include installing an enhanced flow system shallow (12 to 18 inches), a LPD system as shallow as 12 inches, the use of drip disposal installed shallow (12 to 18 inches), elevated sand mounds and pretreatment. The selection of a methodology depends upon the severity of the problem.

When at least 12 inches of stand-off can be maintained by installing a system between 12 and 18 inches from the ground surface, then either low pressure or drip disposal, alone or with pretreatment, is an appropriate solution. When a water table occurs between 18 and 24 inches in a highly permeable (<30 mpi) soil, an elevated sand mound is an appropriate solution to provide treatment and disposal.

When the water table is located within 18 inches of the ground surface, pretreatment must be considered. Either a recirculating sand filter or a lined intermittent sand filter will improve the level of treatment. The method of disposal should be selected to utilize as much aerobic soil treatment as possible. Consideration should be given to the method most likely to provide reliable disposal. Hence, enhanced flow, LPD or drip disposal should be seriously considered. Whenever possible, set-back distances to wells, shellfish water, lakes, ponds and streams should be increased beyond the minimum to allow for reduced soil treatment efficiency. One hundred feet or more to surface water should be practiced where ever lot size allows.

Slow percolation rates

Soils with slow percolation rates as their only problem are few and far between. Normally, soils with slow rates have another problem, such as a restrictive layer or plastic properties that contribute to, or are causing the slow rate problem. The primary solution to soils with rate problems are increasing the drainfield size and improving effluent distribution. Hence, either LPD or drip disposal will enhance the effectiveness of a repair. They also respond very well to water use management and reduction.

The specialist considering a repair in soils with slow percolation rates should evaluate the history of water use in the dwelling. Has there been any recent change due to new occupants, growing family etc., with a resultant change in how much water has been used? How long the system has been operating satisfactorily under the present water usage? This information can assist in sizing a new field. By comparing the soil and water use estimates between the old drainfield and the new site, an attempt can be made to design a system with a 15 to 25 year life expectancy. Such a design should include permanent water saving fixtures and at least enhanced flow distribution. Low pressure distribution and drip disposal are options that are especially well suited to slow rate soils.

Plastic clays

Plastic clay soils have no onsite wastewater disposal solutions. This is a situation where failures are managed not repaired. Strict water conservation will help more than any other solution. Pump and haul is an expensive option that a few may find satisfactory. Separating gray water from black water and pumping and hauling the black water, while putting the gray water into a drainfield may reduce these costs will also reducing health risks associated with a failing drainfield. Repairs attempted in plastic soils should generally be made shallow, in the least plastic horizon, and maximize the effects of evapotransporation. Additionally, homeowners should be advised that the probability of successfully repairing such a system is low. Every effort should be made toward managing the problem as correction is not a likely outcome.

Soil clogging

Soil clogging, organic mat formation, or creeping failure are all evidence of an old system, an undersized system, or both. If sufficient area is available, these systems are usually easy, albeit expensive to repair. Drainfield replacement in the normal solution. Where soil clogging is the only problem, the specialist should evaluate the size and condition of the septic tank and tees before considering their reuse. The age of the system and the patterns of water use (as discussed under slow rates) should also be considered and compared to the new site when designing a repair.

Whenever possible, repairs to a clogged system should be in a manner that preserves the original system. Given sufficient time to rest, most of these failed systems will recover sufficiently to be usable, to some degree, in the future. Installation of a flow diversion valve and the installation of a new system (and probable expansion of the old system) are suggested.

When sufficient area is not available to replace a system, a repair can be made by "going between" the existing laterals. Prior to attempting a repair, the system should be allowed to rest for as long as possible. This will help prevent soil smearing and make for better (but not necessarily pleasant) working conditions when installing the system.

It is rarely possible to repair a system "between the laterals" without encountering portions of the old system. Therefore consideration of alternating between the old and new systems probably will not be possible. If the soils are sufficiently deep, it is recommended that when repairing between laterals, that the repair be installed deeper than the original system. This will tend to avoid areas in the soil where the original system has caused soil clogging and chemical reduction of the soil (evidenced by gleying and organic staining) and potentially reduced infiltration rates (Allison, 1947 and Daniel and Bouma, 1974). Even six to twelve inches additional depth will provide some benefit in this respect.

Restrictive horizons

The variety of pans, discontinuities, impervious layers and restrictions sometimes seems infinite and at odds with successful treatment and disposal objectives. Overcoming the limits of soil restrictions takes outstanding skills if one is to be consistently successful.

The best way to visualize a restriction is not as an impervious layer but rather as a soil horizon with severely restricted water flow through the boundary. When a satisfactory horizon below the restriction exists, the solution to the problem is easy. Install the drainfield below the restriction and anchor a french drain to the restriction above the drainfield. Unfortunately, most problems are not so simple.

When no satisfactory horizon exists below the restriction, the specialist should design a system that minimizes the actual loading rate applied to the **restrictive layer** (as opposed to the trench bottom). This is achieved by maintaining the greatest possible separation distance between the trench bottom and the drainfield, reducing the loading rate applied to the trench bottoms (i.e., over-sizing the system to accommodate for reduced permeability of the restriction), using drip disposal or LPD and potentially modifying trench spacing.

Consider the effluent leaving the trenches. As it moves into the soil it spreads out and is affected by gravity. On all sites the effluent tends to move downward and outward. On sloping sites the downward component of movement includes a downslope vector. The objective when designing a repair for these soils is to allow the effluent to move outward sufficiently, so that when it encounters the restrictions, the effluent application rate does not exceed the permeability of the restriction.

On sloping sites, the downslope component of movement will cause the soil beneath each successive lower lateral to have higher loading rate than the laterals upslope. To counter act this effect, the separation distance between laterals may be increased for the lower laterals. This reduces the application rate to the restrictive horizon and helps reduce the chance of a failure in one of the lower laterals due to what essentially amounts to water mounding.

Chapter 5

Component Functions

Pretreatment Methods and Appurtenances

Septic Tank

Description

The septic tank functions to remove large solid particles from the effluent before it passes to the absorption field. This is accomplished by physical settling of solids, floating of grease and through anaerobic decomposition. In order for these processes to occur efficiently, several conditions must be met.

The tank must be of sufficient size and appropriately shaped to allow settling to occur. Typically length to width ratios are nominally 2:1 and width to depth ratios are nominally 1:1. The Sewage Handling and Disposal Regulations allow for nominal length to width ratios ranging from 2:1 to 3:1 and width to depth ratios of 1:1.

Tanks are normally constructed out of 3,000 psi concrete with either reinforcing wire or synthetic fiber. The material the tank is constructed of from is important only insofar as it is resistant to the corrosive action of the wastewater, that it be watertight, that it is strong enough to support bearing loads placed on it, and that it will not "float" under adverse high water table conditions.

Decomposition in the septic tank, of the organic material present in the wastewater, is a natural process. Decomposing bacteria are introduced to the system through normal use. Therefore the use of additives is not recommended. Additives can be detrimental and have not been shown to be beneficial to the long term process of settling and decomposition.

Application

The septic tank is the most common method of providing pretreatment in onsite wastewater disposal practice. The method is simple, low in cost, requires minimal maintenance and is somewhat effective at keeping solids out of the absorption area.

In order for efficient anaerobic decomposition to occur bacteria must have a favorable environment and a food source. This is not typically how most of us envision conditions inside a septic tank. Fortunately, there are organisms that thrive on wastewater constituents and find the septic tank a hospitable environment. These organisms, which are generally not disease causing, consume and compete with other harmful organisms found in the wastewater. They also convert most of the solid materials to carbon dioxide, water and methane gas. Because they are not 100% efficient, solids accumulate in the bottom of the tank and must be removed periodically.

Failure to pump a tank out frequently enough can permit solids to pass to the drainfield where they will clog soil pores. Eventually this will lead to a failure of the system. The frequency that a given tank will need to be pumped will vary with use. The size of the tank, the organic loading rate, the hydraulic loading rate, and use of chemicals that reduce biological activity will all affect the frequency with which a tank will need to be pumped.

Most of these conditions are beyond practical control after a system has been permitted; hence proper septic tank sizing and design is important. The organic loading rate will be determined by the number of people using the system and the presence or absence of a garbage disposal. When a garbage disposal is present, relatively large amounts of cellulose will be added to the system. Cellulose is fairly resistant to decomposition in the septic tank and will result in the accelerated accumulation of solids in the tank. Generally, under "average use" a septic tank should be pumped every three to five years. When a garbage disposal is used, the tank should be pumped annually.

The only readily controllable aspects affecting the environment in a tank after it has been installed are what is added to the tank. Organic constituents have been discussed and are not readily controllable. The hydraulic loading rate and the addition of chemicals, harmful to the organisms in the tank, can be controlled to some degree through the use of flow reduction devices and owner awareness. The use of laundry detergents, bleach and household cleaners in "normal" amounts is usually not a problem. The use of excessive amounts of these compounds, or of water flow in excess of design capacity can cause problems.

Advantages

The septic tank, as previously noted is a passive design, low in initial cost and in maintenance.

Disadvantages

The septic tank is rarely considered more than 60% effective at reducing solids in the waste flow. There is essentially no reduction in BOD or fecal coliforms.

Dual septic tanks and multiple compartment tanks

The disadvantages and application of dual septic tanks are essentially identical to a single tank. Costs are approximately doubled over a single tank installation but efficiency is increased. In short, the longer the retention time for settling the better. Multi compartment septic tanks are require less excavation and may be less expensive than dual tanks. Multi compartment tanks are at least as effective at removing solids as dual tanks.

The installation of two septic tanks in series (or multi compartment tanks) has been used as a strategy to reduce the amount of suspended solids which reach the drainfield. The first tank provides the major portion of the settling and the second tank gives additional settling time for the remaining lighter solids which did not settle out in the first tank. Dual tanks provide protection against turbulence and surges which may cause solids to move out into the drainfield. If the hydraulic loading of the system has exceeded the ability of the septic tank to provide an adequate retention time a second tank in series should be added. The first tank must provide a minimum storage volume greater than 50% of the daily water flow.

Sand Filters

Description

Sand filters may be broken down into three different types of designs; gravity flow or trickling sand filters, intermittent or dosed sand filters and recirculating sand filters. All three are capable of giving good to excellent treatment with each having its own unique advantages and disadvantages. Trickling sand filters are the simplest design, relying on gravity flow to apply effluent. They are also generally the largest in size and the design most subject to problems with channelized flow. Intermittent sand filters rely on pumps or siphons to dose the septic tank effluent onto the filter. They are substantially smaller in size than gravity flow sand filters and are generally considered to be very reliable in terms of effluent quality. The reduced size is a function of an increased hydraulic loading rate which is made possible because effluent is applied more efficiently to the filter. Recirculating sand filters are the most compact design of the three. Again the size reduction is a function of loadings which are higher for recirculating sand filters than either of the other two designs. A portion of the filtered effluent is returned back to the pump chamber to be recycled through the sand filter again. Properly designed and sized, recirculating sand filters can yield excellent treatment results.

Application

The primary, if not exclusive, function of a sand filter applied to repair a failing onsite system is to provide additional effluent treatment. This objective should always be considered when soil properties do not lend themselves to providing adequate treatment.

Advantages

Effluent from the sand filter, without disinfection, will show a reduction in coliform organisms of from 3 to 4 logs and BOD and suspended solid reductions down to 30 mg/l each or less.

Disadvantages

About the only disadvantage to adding a sand filter to a system is the increase in cost and complexity. Most sand filter systems require a pump which adds to the cost and reduces consumer acceptance of the system.

Aerobic Pretreatment

Description

Aerobic pretreatment in this context is used to refer to mechanical aeration of effluent by an aerobic treatment unit (ATU). Aerobically treated effluent has been proposed to rejuvenate drainfields which have failed due to organic clogging. They have also been proposed as a method of pretreating effluent to reduce BOD and improve the disposal capacity of the soil and as a means to reduce the odor and nuisance associated with a failing system.

Application

Little or no published literature exists to support these potential benefits.

Constructed Wetlands

Description

Constructed wetlands, as used in the context of this manual, are shallow, gravel filled beds, planted with wetland species, which are designed to treat effluent. They are a developing technology and much is yet to be learned about their operation, effectiveness and appropriate application.

Two major design philosophies exist at this time. A pioneer in the constructed wetland concept is B. C. Wolverton, Ph.D. He proposed a design which consisted of one or more, long narrow trenches, 18 to 24 inches deep, planted with Calla lilies or other species. Effluent quality was excellent with reported BOD and SS values often well below 10 mg/l. The design was adapted by local environmental health specialists in Mississippi (where Wolverton did his research) and subsequently spread to other states.

The Tennessee Valley Authority (TVA) published a report (see appendix) describing a modification of the Wolverton design. The TVA design uses shorter, wider beds to reduce the likelihood of clogging by organic matter. Preliminary data indicate that these systems are less likely to clog than the Wolverton design but do not produce the effluent quality that can be expected from the NASA design. It appears to be a classic case of trade-offs. One can have high quality effluent and reduced reliability or vice-versa. Alternatively, the two design concepts may be able to be merged achieving both reliability and treatment efficiency. But not without a new trade-off; cost. Combining designs appears to increase the system cost in proportion to the complexity of the design.

Application

Constructed wetlands (CW) are a developing technology which may have potential to provide pretreatment in soils where adequate disposal capacity exists but treatment capacities are low. Situations where a failing system is located in permeable soils with a high water table, this system may have potential to provide adequate pretreatment to make a repair feasible.

Advantages

The primary advantage of the wetland design is its passive treatment concept. Except for harvesting plants annually, there is little user maintenance. Treatment occurs in the system without mechanical parts to break down and without user input. The system also appeals to environmentally sensitive individuals as a design that is inherently in concert with nature.

Disadvantages

As with any developing technology, many design and performance parameters are unknown. The life expectancy, long term maintenance requirements and long term performance are all unknown at this time.

Tees

Description

Each septic tank should be fitted with both inlet and outlet tees. Tees today are typically schedule 40 PVC or cast in place with the septic tank. Older tees were often cast iron, a material with a limited life expectancy in a septic tank. The tees have four purposes. They reduce the amount of floating solids leaving the septic tank; they vent gasses back through the house plumbing to the vent(s) located of the roof of the house, the help maintain the quiescent nature of the tank by baffling flow into the tank and they create grease holding capacity within the tank.

The tee on the inlet side generally extends into the septic tank effluent to a depth of one foot while the outlet tee generally extends to a depth of one-third the liquid depth (nominally 18 inches) of the tank. Both tees extend eight to ten inches above the liquid level in the tank to provide grease holding capacity.

Zabel filter

The Zabel Multi-Purpose Filter is a commercially available product on the market designed to reduce the amount of solids which will leave the septic tank. The manufacturer claims at least a 67% reduction over other methods of retaining solids. The filter is placed in the septic tank where an outlet tee would normally be installed. Over a period of time the filter will need cleaning. This can be accomplished by rinsing it with high pressure water (a garden hose with a spray nozzle).

The filter may help protect LPD's from orifice plugging due to hair and other suspended solids. Research has shown that most BOD from a septic tank is dissolved so a mechanical filter will not reduce organic loading. The soil must therefore treat the same amount of water and waste load.

Vent Pipe

Description

The vent pipe (or pipes) is part of the plumbing system. It is designed to assure that normal atmospheric pressure is maintained throughout the system of drain pipes in the house. Without proper venting, under some circumstances, a vacuum would be created which would completely stop the flow of water through the drain. Noxious gasses, mostly from the septic tank and sewer line, are also vented up to the roof where they are released.

Odors from the vent system are rarely (but not never) a problem. When they are a problem there are several possible solutions. On newer septic systems (less than 3 years old and often less than a year) which are significantly underloaded objectionable odors may occur. Odors may also occur when atmospheric conditions are such that the vent gasses settle to the ground near the stack instead of blowing away. This is typically a short term condition and requires no remedy.

Final Treatment and Disposal Methods and Appurtenances

Drainfield

Description

The drainfield is the single most important element in the wastewater system. While the other elements can be manipulated in the design and planning phases to match the user's needs, man cannot effectively alter the quality of the soil on the site. Even after a system is installed, other components can be modified or replaced. The soil, where most of the effluent treatment and disposal actually occurs, cannot be altered. Systems must be designed to utilize the best soil properties and accommodate for the worst soil properties; otherwise another failure is inevitable.

The drainfield is composed of a series of trenches, installed on contour, up to 100 feet long and usually 2 or 3 feet wide. The trench depth is determined based on soil permeability, depth to restrictive horizons and depth to other treatment limiting factors such as rock or water table. The trench normally has 13 inches of stone installed with a 4 inch perforated pipe running the length of the trench nominally 1 to 2 inches below the top of the gravel.

Effluent leaves the pipe near the beginning of the trench, enters the gravel and follows the trench bottom until it is absorbed into the soil. A biologically active zone is created at the interface between the gravel and the soil. This zone, often referred to as an organic (and less often as a zoogleal mat), acts as both a biological filter and a restrictive layer.

Application

The ideal drainfield application is to sites where 2 to 4 feet of unsaturated soil exists below the trench bottom to complete the process of renovating the effluent. In reality this is rarely the case with new construction and is even less likely to occur in repair situations. In practice, 18 inches (or more) of unsaturated soil should be present for treatment. In many repair situations there will be significantly less than 18 inches of suitable soil with which to work. When this stand-off distance cannot be met, a drainfield can also be used in concert with other technologies, such as various pretreatment schemes or shallow placement methods in order to achieve equivalent treatment.

Advantages

The drainfield is by far the most common method of wastewater treatment and disposal. It is cost effective, environmentally safe when proper site conditions are met, and contractors abound who can competently install this type of system.

Disadvantages

Many sites are inappropriate for a conventional drainfield. Water table, rock, restrictive horizons or other limiting factors may preclude a drainfield from both treating and disposing of sewage. This can result in wells becoming contaminated or sewage surfacing on top of the ground.

Dosed systems conventional (enhanced flow)

Description

Enhanced flow technology is nothing more than a conventional septic tank and drainfield system, sized and sited according to current regulations, that has effluent applied in a manner designed distribute effluent more evenly between laterals. The only additional appurtenance is a conventional pump chamber with a float control system and an audio visual alarm. The pump and pump chamber, in some instances, may be somewhat larger than a conventional pump chamber used only to overcome gravity.

Application

Dosed systems are most appropriate where the intermittent application of effluent will assist in either the treatment or disposal of effluent. Soils with slow percolation rates will benefit the most.

Advantages

Enhanced flow is a simple technology offering modest gains in optimizing effluent distribution. Contractors are familiar with construction techniques and homeowner acceptance should not present problems. Where cost is a factor, a shallow placed, enhanced flow system may provide an acceptable compromise when compared to LPD.

Disadvantages

Aside from the additional cost of a pump and the modest reduction in system reliability v.s. systems without a pump, enhance flow systems have no significant disadvantages.

Low Pressure Distribution

Description

Low pressure distribution (LPD) systems are essentially just a modified drainfield. Effluent is conveyed to the absorption area of an LPD system by pump rather than gravity. The absorption trenches have 1.25 to 1.5 inch PVC pipe with small diameter holes (3/16" to 1/4"). The effluent from the pump pressurizes a manifold that distributes the effluent evenly throughout the piping in the laterals.

By design, the LPD system is intended to provide nearly equal amounts of effluent to each lateral as well as assuring that the entire length of each lateral is used. In contrast, a conventional gravity flow drainfield typically has 30% to 50% flow variation between laterals when installed and has no provision for distributing effluent evenly along the trench length. LPD systems are designed to eliminate ponding along the trench bottom. This provides for aerobic conditions in the trench to maximize the rate of treatment of the effluent. Where sustained ponding occurs, the system is undersized.

Application

Low pressure distribution systems lend themselves to truly marginal soils. These are soils that are not deep enough in terms of depth to rock or water table for a conventional system or are just barely deep enough but for one reason or another require special precautions. Soils with slower than average percolation rates can also be used successfully. Under no circumstances should LPD technology be applied where plastic soils are encountered. The even distribution of effluent wets the soil causing swelling and effluent ponding begins almost immediately.

Advantages

LPD systems offer the ability to make installations as shallow as 12 inches and thereby maintain or increase separation distances to limiting soil features. Experimentally, LPD has been installed at original surface grade successfully. Gravel "trenches" are built on a plowed surface, the laterals and manifold are installed and the system covered deep enough to prevent freezing or physical damage. The cost of LPD systems is moderate but generally not considered excessive. Additionally, because of their theoretical longevity, (due to dosing and even distribution) they may prove to be a better value (cost per unit time) than most systems.

Disadvantages

The primary disadvantages of LPD systems are the increased cost compared to a conventional system, the potential for pump failure and plugging of the laterals and throttling valves. LPD systems require routine maintenance after the first three to five years.

LPD's must be designed for each specific site by a qualified designer. No two systems are alike. Proper placement, controls and pump selection are critical. Pump replacement in the future is also critical to make sure the system operates as designed.

Elevated Sand mounds

Description

Elevated sand mounds (ESM) dispose of septic tank effluent by applying it to a sand bed built over natural soil. The sand media provides effluent treatment prior to its disposal into the natural soil for final treatment and disposal.

Application

Elevated sand mounds are most appropriate where disposal issues are not a concern but the existing soils are not capable of providing adequate treatment before allowing the effluent to enter the groundwater or fractured rock. Ideally, the combination of sand in the mound (12 to 24 inches) plus the underlying suitable soil (18 to 60 inches, or more) will provide for satisfactory effluent renovation. In all repair situations this should be the goal. Unfortunately this goal cannot always be realized.

Advantages

The primary advantages of the ESM is its ability to treat and dispose of septic tank effluent in a small area. Mounds are not however, a panacea for all sites that are unsuited for drainfields.

Disadvantages

The primary disadvantage of a mound is the cost. The expense varies with locality but $7,000 to $10,000 cost estimates are commonly cited. Additionally, another less significant disadvantage is that the ESM utilizes a pump. Pumps will eventually need to be replaced and are considered as a disadvantage by some individuals.

Additionally, improperly sited mounds frequently leak at the base of the mound. Most drainfield failures occur on sites that are not particularly well suited for a mound. The specialist should use caution when establishing a repair strategy using a mound on a poorly suited site. At a minimum the homeowner should be aware of the costs and potential for leakage. The specialist should also document that the expected result will be an improvement over the existing conditions. Frequently the wastewater from a leaking mound is low in fecal organisms and may well represent an improvement over a failing drainfield. Simply put, don't consider the mound as a panacea and do not over sell it on marginally suited sites. On sites meeting the criteria for siting a mound, they are an effective and appropriate repair methodology.

Drip disposal or "trickle" irrigation

Description

Trickle irrigation is a developing technology, based on using flexible irrigation tubing to dispose of effluent. The tubing used is flexible plastic pipe with emitters installed at set intervals. The emitters drip at a fairly constant interval over a wide range of pressures. Unlike LPD design, pressure variation is usually of no concern. Some manufacturers have been installing drip systems for over five years. It has been successfully used for a number of years in agriculture.

These systems are basically designed to be high pressure distribution fluid handling systems. The effluent pumped into the drip tubes must be free of solids to avoid clogging the emitters. At least two different pretreatment schemes have been developed to accomplish this. Several manufacturers use aerobic pretreatment to remove solids. Another manufacturer is filtering septic tank effluent through a proprietary disk filter. Much like LPD's, these systems need to be designed for each specific site.

Application

This technology may be applicable in the same situations where a LPD would be appropriate.

Advantages

The cost of operating a trickle irrigation system are not yet known. Methods of installing the pipe vary and can involve the use of a conventional backhoe or in some instances use a chisel plow. In the latter instance, site disturbance is minimal compared to a backhoe.

Disadvantages

Mostly unknown at this time. Where ATU's are employed, maintenance similar to that required by the Discharging Regulations must be employed. The cost of materials (w/o an ATU) has been given as approximately $3,000 by one company. It appears then that this technology will be competitive with LPD systems.

Distribution Box

Description

The distribution box has a single purpose: to divide effluent flows evenly among absorption field laterals. Unfortunately, the distribution box is not an efficient or effective method of achieving equal flow splitting.

Ideally the distribution box should be placed on a concrete pad or otherwise solidly anchored to prevent it from tilting or shifting in place. The use of Speed Levelers™ or Dial-a-Flows™ facilitates future adjustments. Because of their constant radius and ease of adjustment, these devices most likely improve the accuracy with which a box can be leveled initially and in the future.

Application

The distribution box may be used anywhere effluent is to be split between multiple laterals and even distribution is not critical. Most new drainfields fall into this category where the soils are clearly satisfactory. When repairing a failing system better than average distribution may be necessary to assure that no portion of the soil absorption system is overloaded.

Advantages

The primary advantages of the distribution box are its low cost, ease of installation, and contractor familiarity.

Disadvantages

The distribution box is unquestionably the weak link in any gravity flow system. Minor shifts in elevation will alter flow patterns in a gravity flow distribution box significantly. The most serious problems occur when a distribution box is tilted to allow excess flow to the lowest lateral.

Flow diversion valves

Description

A flow diversion valve is a device installed in the effluent line between the septic tank and the distribution box. It requires the use of two absorption fields, each normally equal to half the required absorption field area. When dealing with repairs in marginal and unsuitable soils, larger absorption areas may be appropriate. The flow diversion valve is a manually operated device that directs flow exclusively to one absorption field. Periodically (usually annually) the valve is switched and the second field is placed into use while the first one is allowed to rest.

Application

Flow diversion valves do not achieve unsaturated flow like LPD or drip disposal but it does provide a mechanism to allow for long term wetting and drying periods. When there is adequate area to replace a drainfield entirely and the soils are not entirely satisfactory, a flow diversion valve may be installed to allow the homeowner to alternate between the old and the new systems. At least one year, and preferably several years should be allowed to pass before reusing the old system.

When a failing system is repaired "between the lines" there is often an urge to use a flow diversion valve. If this is to be done, extreme care must be taken during installation of the repair to assure that the old system is not intersected. If this occurs, it will not be possible to maintain hydraulic separation of the two systems and using a flow diversion valve will accomplish little or nothing. Realistically, installing a 3 foot wide trench between two other 3 foot wide trenches, 9 feet on center without knowing the precise location of the original trenches will be impossible.

Advantages

Flow diversion allows for long term periods of wetting and drying which allows soils to renew their absorptive capacity. While apparently not as effective as LPD or drip disposal, the concept is very inexpensive to apply. Materials are readily available and competent installation is widely available.

Long term resting has been proven to be the only way to rejuvenate a drainfield. Resting can at least partially reverse damage due to both organic overload and hydraulic overload.

Diversion valves can be successfully used on slower soils to rejuvenate each half on an annual basis. Plus, in the event of failure, a standby system is already in place. A properly sized and installed gravity system will easily last several years on 1/2 of the design. Providing long term resting for gravity systems is the best available strategy for making conventional systems a permanent solution to onsite wastewater treatment and disposal.

Disadvantages

The only practical disadvantage of using a flow diversion valve is assuring that they are routinely switched to allow both fields to be used.

Privies and Blackwater Treatment Devices

Composting toilets

Description

Composting toilets compost human waste (along with additional carbonaceous matter that may be needed) and produce a humus like material after six months to a year of biological action. The resulting material, when properly composted, should be safe to dispose of by burying it. This should never be done in vegetable garden. In actuality, new fecal matter is continually added to the composting pile and whatever material is removed should be treated as if it is contaminated with fresh feces.

Application

Composting toilets can be used to treat human black water under the Sewage Handling and Disposal Regulations. The two best advantages of the method are where there is little or no grey water being produced (i.e., certain commercial or recreational uses) and where conditions exist that make it necessary to separate the black and grey water components out and treat them separately. Generally the C:N ration should be approximately 25:1. Consequently it is frequently necessary to add carbon to make up for excess nitrogen. Shredded newspaper or kitchen garbage (also shredded) can be used to balance the C:N ratio.

Advantages

The primary advantage of composting toilets is that their use and functioning are entirely independent of site and soil conditions.

Disadvantages

Disadvantages to the systems are public acceptance, cost, difficulty installing in existing structures, odors and problems maintaining a carbon to nitrogen ratio (C:N) suitable for good composting action. Composting toilets are expensive when one considers that they represent only a portion of the wastewater disposal solution. In some instances they have been reported to cause objectionable odor problems. Mechanical ventilation kits for these systems can reduce or eliminate this complaint. Installation is difficult or impossible for existing construction and the composting chamber, which resides one floor below the toilet, requires substantial space. Either a basement is needed (for one story homes) or first floor space must be given up when the toilet is installed on a second floor. Finally, many individuals find it difficult adjusting to having an indoor privy. Not flushing and having to feed the system a carbon source amounts to a lifestyle adjustment that is difficult to make.

Vault privies

Vault privies are privies built over a septic tank or other type of vault. Their purpose and applications are similar to those of a composting toilet but are most useful where an outdoor privy is acceptable. Vault privies are periodically pumped out to remove accumulated organic matter.

References

Allison, L.E. 1947. Effect of microorganisms on permeability of soil under prolonged submergence. Soil Sci. 63: 439-450

Avnimelech, Y. and Z. Nevo, 1964. Biological clogging of sands. Soil Sci. 98:222-226.

Baum, S.J. 1982. Introduction to organic and biological chemistry. Macmillian Publishing Co., Inc., New York.

Daniel, T.C. and J. Bouma. 1974. Column studies of soil clogging in a slowly permeable soil as a function of effluent quality. J. Environ. Qual. 3: 231-326.

Kristiansen, R. 1982. The soil as a renovating medium. The fate of pollutants in soil-organic material. In A.S., and R.W. Seabloom (eds.) Alternative Wastewater Treatment. D. Reidel Publ. pp. 121-128.

Reneau, R.B., Jr., and D.E. Petry. 1975. Movement of coliform bacteria from septic tank effluent through selected coastal plain soils of Virginia. J. Environ. Qual. 4: 41-44.

Discharging Wastewater Treatment Technologies

Sand Filters

Technical Services

Virginia Department of Health
Office of Environmental Health Services

PREFACE

The following report on sand filter technology was prepared as part of a training program for the Alternative Discharging Sewage Treatment System Regulations for Single Family Homes. The contents of this report were taken directly from the Environmental Protection Agency Publication entitled: Design Manual - Onsite Wastewater Treatment and Disposal Systems (October, 1980). The information in this EPA publication was used as a basis for sand filter training because it covers many aspects of sand filter design, construction, and use.

At the present time, "generic" plans and specifications for sand filter systems have not been developed by the Division of Onsite Sewage and Water Services. However, such generic plans will substantially follow the design standards and recommendations provided in the EPA Onsite Manual, as reproduced in this report. Until such time as "generic" plans and specifications are provided by the Division of Onsite Sewage and Water Services, the EPA manual should be used to guide you in the plan review process.

David Effert,
Technical Services Chief
July 24, 1992

Intermittent Sand Filters[1]

Introduction

Intermittent sand filtration may be defined as the intermittent application of wastewater to a bed of granular material which is underdrained to collect and discharge the final effluent. One of the oldest methods of wastewater treatment known, intermittent sand filtration, if properly designed, operated, and constructed, will produce effluent of very high quality. Currently, many intermittent sand filters are used throughout the United States to treat wastewater from small commercial and institutional developments and from individual homes. The use of intermittent sand filters for upgrading stabilization ponds has also become popular.

Intermittent sand filtration is well suited to onsite wastewater treatment and disposal. The process is highly efficient, yet requires a minimum of operation and maintenance. Normally, it would be used to polish effluent from septic tank or aerobic treatment processes and would be followed by disinfection (as required) prior to reuse or disposal to land or surface waters.

Description

Intermittent sand filters are beds of granular materials 24 to 36 inches (61 to 91 cm) deep and underlain by graded gravel and collecting tile. Wastewater is applied intermittently to the surface of the bed through distribution pipes or troughs. Uniform distribution is normally obtained by dosing so as to flood the entire surface of the bed.

Filters may be designed to provide free access (open filters), or may be buried in the ground (buried filters). A relatively new concept in filtration employs recirculation of filter effluent (recirculating filters).

The mechanisms of purification attained by intermittent sand filters are complex and not well understood even today. Filters provide physical straining and sedimentation of solid materials within the media grains. Chemical sorption also plays a role in the removal of some materials. However, successful treatment of wastewater is dependent upon the biochemical transformations occurring within the filter. Without the assimilation of filtered and sorbed materials by biological growth within the filter, the process would fail to operate properly. There is a broad range of trophic levels operating within the filter, from the bacteria to annelid worms.

---

[1]This material has been taken from the Environmental Protection Agency Design Manual for Onsite Wastewater Treatment and Disposal Systems. U.S. Environmental Protection Agency, Office of Water Programs Operation, Office of Research and Development, Municipal Environmental Research Laboratory. Cincinnati, OH. EPA 625/1-80-012. October 1980. pgs. 113-140.

Since filters entrap, sorb, and assimilate materials in the wastewater, it is not surprising to find that the interstices between the grains may fill, and the filter may eventually clog. Clogging may be caused by physical, chemical, and biological factors. Physical clogging is normally caused by the accumulation of stable solid materials within or on the surface of the sand. It is dependent on grain size and porosity of the filter media, and on wastewater suspended solids characteristics. The prescription, coagulation, and adsorption of a variety of materials in wastewater may also contribute to the clogging problem in some filter operations. Biological clogging is due primarily to an improper balance of the intricate biological population within the filter. Toxic components in the wastewater, high organic loading, absence of dissolved oxygen, and decrease in filter temperatures are the most likely causes of microbial imbalances. Accumulation of biological slimes and a decrease in the rate of decomposition of entrapped wastewater contaminants within the filter accelerates filter clogging. All forms of pore clogging likely occur simultaneously throughout the filter bed. The dominant clogging mechanism is dependent upon wastewater characteristics, method and rate of wastewater application, characteristics of the filtering media, and filter environmental conditions.

Application

Intermittent sand filtration is well adapted to onsite disposal. Its size is limited by land availability. The process is applicable to single homes and clusters of dwellings. The wastewater applied to the intermittent filters should be pretreated at least by sedimentation. Septic tanks should be required as a minimum. Additional pretreatment by aerobic biological processes normally results in higher acceptable rates of wastewater application and longer filter runs. Although extensive field experience is lacking to date, the application of pretreated graywaters to intermittent sand filters may be advantageously employed. There is some evidence that higher loading rates and longer filter runs can be achieved with pretreated graywaters.

Site constraints should not limit the application on intermittent sand filters, although odors from open filters receiving septic tank effluent may require isolation of the process from dwellings. Filters are often partially (or completely) buried in the ground, but may be constructed above ground when dictated by shallow bedrock or high water tables. Covered filters are required in areas with extended periods of subfreezing weather. Excessive long-term rainfall and runoff on submerged filter systems may be detrimental to performance, requiring appropriate measures to divert theses sources away from the system.

Factors Affecting Performance

The degree of stabilization attained by an intermittent sand filter is dependent upon:

(1) the type of biodegradability of wastewater applied to the filter,
(2) the environmental conditions within the filter, and
(3) the design characteristics of the filter.

Reaeration and temperature are two of the most important environmental conditions that affect the degree of wastewater purification through an intermittent sand filter. Availability of oxygen within the pores allows for the aerobic decomposition of the wastewater. Temperature directly affects the rate of microbial growth, chemical reactions, adsorption mechanisms, and other factors that contribute to the stabilization of wastewater within the sand media.

Proper selection of process design variables also affects the degree of purification of wastewater by intermittent filters. A brief discussion of those variables is presented below.

Media Size and Distribution

The successful use of a granular material as a filtering media is dependent upon the proper choice of size and uniformity of the grains. Filter media size and uniformity are expressed in terms of "effective size" and "uniformity coefficient." The effective size is the size of the grain, in millimeters, such that 10% by weight are smaller. The uniformity coefficient is the ratio of the grain size that has 60% by weight finer than itself to the size which is 10% finer than itself. The effective size of the granular media affects the quantity of wastewater that may be filtered, the rate of filtration, the penetration depth of particulate matter, and the quality of the filter effluent. Granular media that is too coarse lowers the retention time of the applied wastewater through the filter to a point where adequate biological decomposition is not attained. Too fine a media limits the quantity of wastewater that may be successfully filtered, and will lead to early filter clogging. This is due to the low hydraulic capacity and the existence of capillary saturation, characteristic of fine materials. Metcalf and Eddy and Boyce recommended that not more than 1% of the media should be finer than 0.13 mm. Recommended filter media effective sizes range from a minimum of 0.25 mm up to approximately 1.5 mm. Uniformity coefficients (UC) for intermittent filter media normally should be less than 4.0.

Granular media other than sand that have been used include anthracite, garnet, ilmenite, activated carbon, and mineral tailings. The media selected should be durable and insoluble in water. Total organic matter should be less than 1%, and total acid soluble matter should not exceed 3%. Any clay, loam, limestone, or organic material may increase the initial adsorption capacity of the sand, but may lead to a serious clogging condition as the filter ages.

Shapes of individual media grains include round, oval, and angular configurations. Purification of wastewater infiltrating through granular media is dependent upon the adsorption and oxidation of organic matter in the wastewater. To a limiting extent, this is dependent on the shape of the grain; however, it is more dependent on the size distribution of the grains, which is characterized by the UC.

The arrangement or placement of different sizes of grains throughout the filter bed is also an important design consideration. A homogeneous bed of one effective size media does not occur often due to construction practices and variations in local materials. In a bed having fine media layers placed above coarse layers, the downward attraction of wastewater is not as great due to the lower amount of cohesion of the water in the larger pores. The coarse media will not draw the water out of the fine media, thereby casing the bottom layers of the fine material to remain saturated with water. This saturated zone acts as a water seal, limits oxidation, promotes clogging, and reduces the action of the filter to a mere straining mechanism. The use of media with a UC of less than 4.0 minimizes this problem.

The media arrangement of coarse over fine appears theoretically to be the most favorable, but it may be difficult to operate such a filter due to internal clogging throughout the filter.

Hydraulic Loading Rate

The Hydraulic loading rate may be defined as the volume of liquid applied to the surface area of the sand filter over a designated length of time. Hydraulic loading is normally expressed as gpd/ft^2, or cm/day. Values of recommended loading rates for intermittent sand filtration vary throughout the literature and depend upon the effective size of sand and the type of wastewater. They normally range from 0.75 to 15 gpd/ft^2 (0.3 to 0.6 $m^3/m^2/d$).

Organic Loading Rate

The organic loading rate may be defined as the amount of soluble and insoluble organic matter applied per unit volume of filter bed over a designated length of time. Organic loading rates are not often reported in the literature. However, early investigations found that the performance of intermittent sand filters was dependent upon the accumulation of stable organic material in the filter bed. To account for this, suggested hydraulic loading rates today are often given for a particular type of wastewater. Allowable loading rates increase with the degree of pretreatment. A strict relationship establishing and organic loading rate, however, has not yet been clearly defined in the literature.

Depth of Media

Depths of intermittent sand filters were initially designed to be 4 to 10 feet; however, it was soon realized at the Lawrence Experimental Station that most of the purification of wastewater occurred within the top 9 to 12 in. (23 to 30 cm) of the bed. Additional bed depth did not improve the wastewater purification to any significant degree. Most media depths used today range from 24 to 42 in. (62 to 107 cm). The use of shallow filter beds helps to keep the cost of installation low. Deeper beds tend to product a more constant effluent quality, are not affected as severely by rainfall or snow melt, and permit the removal of more media before media replacement becomes necessary.

Dosing Techniques and Frequency

Dosing techniques refer to methods of application of wastewater to the intermittent sand filter. Dosing of intermittent filters is critical to the performance of the process. The system must be designed to insure uniform distribution of wastewater throughout the filter cross-section. Sufficient resting must also be provided between dosages to obtain aerobic conditions. In small filters, wastewater is applied in doses large enough to entirely flood the filter surface with at least 3 in. (8 cm) of water, thereby insuring adequate distribution. Dosing frequency is dependent upon media size, but should be greater with smaller doses for coarser media.

Dosing methods that have been used include ridge and furrow application, drain tile distribution, surface flooding, and spray distribution methods. Early sand filters for municipal wastewater were surface units that normally employed ridge and furrow or spray distribution methods. Intermittent filters in use today are often built below the ground surface and employ tile distribution.

The frequency of dosing intermittent sand filters is open to considerable design judgement. Most of the earlier studies used a dosing frequency of 1/day. The Florida studies investigated multiple dosings and concluded that the BOD removal efficiency of filters with media effective size greater than 0.45 mm is appreciably increased when the frequency of loading is increased beyond twice per day. This multiple dosing concept is successfully used in recirculating sand filter systems in Illinois, which employ a dosing frequency of once ever 30 min.

Maintenance Techniques

Various techniques to maintain the filter bed may be employed when the bed becomes clogged. Some of these include:

(1) resting the bed for a period of time,
(2) raking the surface layer and thus breaking the inhibiting crust, or
(3) removing the top surface media and replacing it with clean media.

The effectiveness of each technique has not been clearly established in the literature.

Filter Performance

(Sand filter effluent quality can be summarized as follows)... intermittent filters produce high-quality effluent with respect to BOD_5 and suspended solids. Normally, nitrogen is transformed almost completely to the nitrate form provided the filter remains aerobic. Rates of nitrification may decrease in winter months as temperatures fall. Little or no denitrification should occur in properly operated intermittent filters.

Total and ortho-phosphate concentrations can be reduced up to approximately 50% in clean sand; but the exchange capacity of most of the sand as well as phosphorus removal after maturation is low. Use of calcareous sand or their high-aluminum or iron materials intermixed within the sand may produce significant phosphorus removal. Chowdhry and Brandes, et. al., reported phosphorus removals of up to 90% when additions of 4% "red mud" (high in Al_2O_3 and Fe_2O_3) were made to a medium sand. Intermittent filters are capable of reducing total and fecal coliforms by 2 to 4 logs, producing effluent values ranging from 100 to 3,000 per 100 mil and 1,000 to 100,000/100 ml for fecal and total coliforms, respectively.

Design Criteria

Buried Filters

Table 1 summarizes design criteria for subsurface intermittent sand filters.

Hydraulic loading of these filters is normally equal to or less than 1.0 gpd/ft^2 (0.04 m^3/m^2/d) for full-time residences. This value is similar to loading rates for absorption systems in sandy soils after equilibrium conditions are obtained. When filters are designed for facilities with seasonal occupation, hydraulic loading may be increased to 2.0 gpd/ft^2 (0.08 m^3/m^2/d) since sufficient time will be available for drying and restoring the infiltrative surface of the bed.

Table 1. Design Criteria For Buried Intermittent Sand Filters[1]

ITEM	DESIGN CRITERIA
Pretreatment:	Minimum level - sedimentation (septic tank or equivalent)
Hydraulic Loading:	
All year	<1.0 gpd/ft^2
Seasonal	<2.0 gpd/ft^2
Media:	
Material	Washed durable granular material (less than 1 percent organic matter by weight)
Effective size	0.50 to 1.00 mm
Unif. Coeff.	<4.0 (<3.5 preferable)
Depth	24 to 36 inches
Underdrains:	
Material	Open joint or perforated pipe
Slope	0.5 to 1.0 percent
Bedding	Washed durable gravel or crushed stone (1/4 to 1-1/2 in.)
Venting:	Upstream end
Distribution:	
Material	Open Joint or perforated pipe
Bedding	Washed durable gravel or stone (3/4 to 2-1/2 in.)
Venting:	Downstream end
Dosing:	Flood filter; frequency greater than 2 times per day

[1]This material has been taken from the Environmental Protection Agency Design Manual for Onsite Wastewater Treatment and Disposal Systems. U.S. Environmental Protection Agency, EPA 625/1-80-012. October 1980. pg. 124. **Virginia Department of Health design standards have not been established at this time.**

The effective size of media for subsurface filters ranges from 0.35 to 1.0 mm with a UC less than 4.0, and preferably less than 3.5. Finer media will tend to clog more readily, whereas coarser media may result in poorer distribution and will normally produce a lower effluent quality.

Distribution and underdrains are normally perforated or open-joint pipe with a minimum 4-in. (10-cm) diameter. The distribution and underdrain lines are surrounded by at least 8 in. of washed durable gravel or crushed stone. For distribution lines, the gravel or stone is usually smaller than 2-1/2 in. (6 cm) but larger than 3/4 in. (2 cm), whereas the size range of the gravel or stone for the underdrains is between 1-1/2 to 1/4 in. (3.8 to 0.6 cm). Slopes of underdrain pipe range from 0.5 to 1%. With dosing, there would be no requirement for slopes on distribution piping.

Proper dosing to the filter is critical to its successful performance. The dosing system is designed to flood the entire filter during the dosing cycle. A dosing frequency of greater than two times per day is recommended.

Free Access Filters (Non-Recirculating)

Design Criteria for free access filters are presented in Table 2.

Hydraulic loading to these filters depends upon media size and wastewater characteristics. Septic tank effluent may be applied at rates up to 5 gpd/ft^2 (0.2 m^3/m^2/d), whereas a higher quality pretreated wastewater may be applied at rates as high as 10 gal/d ft^2 (40 cm/d). Selection of hydraulic loading will also be influenced by desired filter run times. Higher acceptable loadings on these filters as compared to subsurface filters relates primarily to the accessibility of the filter surface for maintenance.

Media characteristics and underdrain systems for free access filters are similar to those for subsurface filters. Distribution is often provided through pipelines and directed on splash plates located at the center or corners of the sand surface. Occasionally, troughs or spray nozzles are employed as well, and ridge and furrow application has been successful during winter operation in severe climatic conditions. Dosing of the filter should provide for flooding the bed to a depth of approximately 2 in. Dosing frequency is usually greater than two times per day. For coarser media (greater than 0.5 mm), a dosing frequency greater than 4 times per day is desirable.

The properties of the wastewater applied affect the clogging characteristics of the filter and, therefore, the methods of filter maintenance. Dual filters, each designed to carry the design flow rate, may be desirable when treating septic tank effluent to allow sufficient resting after clogging.

Table 2. Design Criteria For Free Access Intermittent Sand Filters[1]

ITEM	DESIGN CRITERIA
Pretreatment:	Minimum level - sedimentation (septic tank or equivalent)
Hydraulic Loading:	
Septic tank feed	2.0 to 5.0 gpd/ft^2
Aerobic feed	5.0 to 10.0 gpd/ft^2
Media:	
Material	Washed durable granular material (less than 1 percent organic matter by weight)
Effective size	0.35 to 1.00 mm
Unif. Coeff.	<4.0 (<3.5 preferable)
Depth	24 to 36 inches
Underdrains:	
Material	Open joint or perforated pipe
Slope	0.5 to 1.0 percent
Bedding	Washed durable gravel or crushed stone (1/4 to 1-1/2 in.)
Venting	Upstream end
Distribution:	Troughs on surface; splash plates at center or corners; sprinkler distribution
Dosing:	Flood filter to 2 inches; frequency greater than 2 times per day
Number of filters:	
Septic tank feed	Dual filters, each sized for design flow
Aerobic feed	Single filter

[1]This material has been taken from the Environmental Protection Agency Design Manual for Onsite Wastewater Treatment and Disposal Systems. U.S. Environmental Protection Agency, EPA 625/1-80-012. October 1980. pg. 126. **Virginia Department of Health design standards have not been established at this time.**

Recirculating Filters

Proposed design criteria for recirculating intermittent sand filters are presented in Table 3. Theses free access filters employ a recirculation (dosing) tank between the pretreatment unit and filter with provision for return of filtered effluent to the recirculation tank.

Table 3. Design Criteria For Recirculating Intermittent Sand Filters[1]

ITEM	DESIGN CRITERIA
Pretreatment:	Minimum level - sedimentation (septic tank or equivalent)
Hydraulic Loading:	3.0 to 5.0 gpd/ft^2 (forward flow)
Media:	
Material	Washed durable granular material (less than 1 percent organic matter by weight)
Effective size	0.3 to 1.5 mm
Unif. Coeff.	<4.0 (<3.5 preferable)
Depth	24 to 36 inches
Underdrains:	
Material	Open joint or perforated pipe
Slope	0.5 to 1.0 percent
Bedding	Washed durable gravel or crushed stone (1/4 to 1-1/2 in.)
Venting	Upstream end
Distribution:	Troughs on surface; splash plates at center or corners; sprinkler distribution
Recirculation Ratio:	3:1 to 5:1 (5:1 preferable)
Dosing:	Flood filter to approx. 2 inches; pump 5 to 10 min per 30 min; empty recirculation tank in less than 20 min
Recirculation Tank:	Volume equivalent to at least one day's raw wastewater flow

[1]This material has been taken from the Environmental Protection Agency Design Manual for Onsite Wastewater Treatment and Disposal Systems. U.S. Environmental Protection Agency, EPA 625/1-80-012. October 1980. pg. 124. **Virginia Department of Health design standards have not been established at this time.**

Hydraulic loading ranges from 3 to 5 gpd/ft^2 (0.12 to 0.20 m^3/m^2/d) depending on media size. Media size range is from 0.3 to 1.5 mm, the coarser sizes being recommended (23)(26). Underdrain and distribution arrangements are similar to those for free access filters. Recirculation is critical to effective operation, and 3:1 to 5:1 recirculation ratio (Recycle: Forward Flow) is preferable.

Pumps should be set by timer to dose approximately 5 to 10 min per 30 min. Longer dosing cycles may be desirable for larger installations - 20 min every 2 to 3 hr. Dosing should be at a rate high enough to insure flooding of the surface to greater than 2 in. (5 cm). Recirculation chambers are normally sized at 1/4 to 1/2 the volume of the septic tank.

Construction Features

Buried Filters

A typical plan and profile of a buried intermittent sand filter are depicted in Figure 1. The filter is placed within the ground with a natural topsoil cover in excess of 10 in. (25 cm) over the crown of the distribution pipes. The filter must be carefully constructed after excavation and the granular fill settled by flooding. Distribution and underdrain lines should be constructed of an acceptable material with a minimum diameter of 4 in. (10 cm). The tile is normally laid with open joints with sections space not less than 1/4 inc. (0.6 cm) or greater than 1/2 in. (1.3 cm) apart. If continuous pipeline is used, conventional perforated pipe will provide adequate distribution and collection of wastewater within the filter.

The underdrain lines are laid to grade (0.5 to 1%) and one line is provided for each 12 ft (3.6 m) of trench width. Underdrains are provided with a vent pipe at the upstream end extending to the ground surface. The bedding material for underdrain lines in usually a minimum of 10 in. (25 cm) washed graded gravel or stone with sizes ranging from 1/4 to 1-1/2 in. (0.6 to 3.8 cm). The gravel or stone may be overlain with a minimum of 3 in. (8 cm) of washed pea gravel (1/4 - to 3/8 in. [1.9- to 6.3-cm] size) is usually employed for bedding of distribution lines. Marsh hay, washed pea gravel, or drainage fabric should be placed between the bedding material and the natural topsoil.

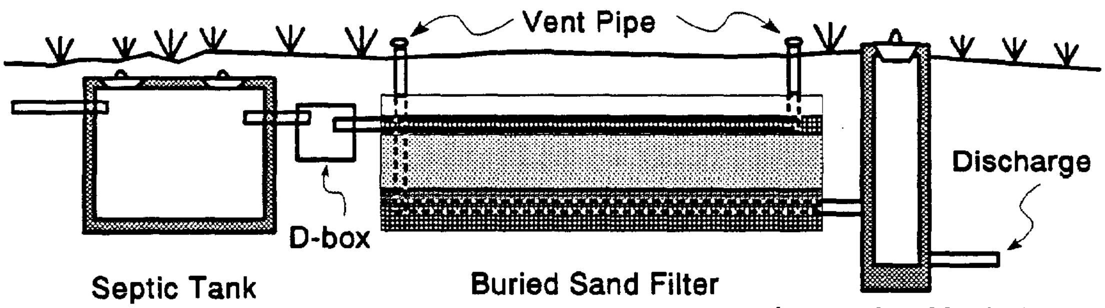
The finished grade over the filter should be mounded so as to provide drainage of rainfall away from the filter bed. A grade of approximately 3 to 5%, depending upon topsoil characteristics, would be sufficient.

Any washed, durable granular material that is low in organic matter may be used for filter medium. Mixtures of sand, slag, coal, or other materials have been used to enhance the removal of selected pollutants and to extend filter life. Care must be taken, however, to insure that the media does not stratify with fine layers over coarse.

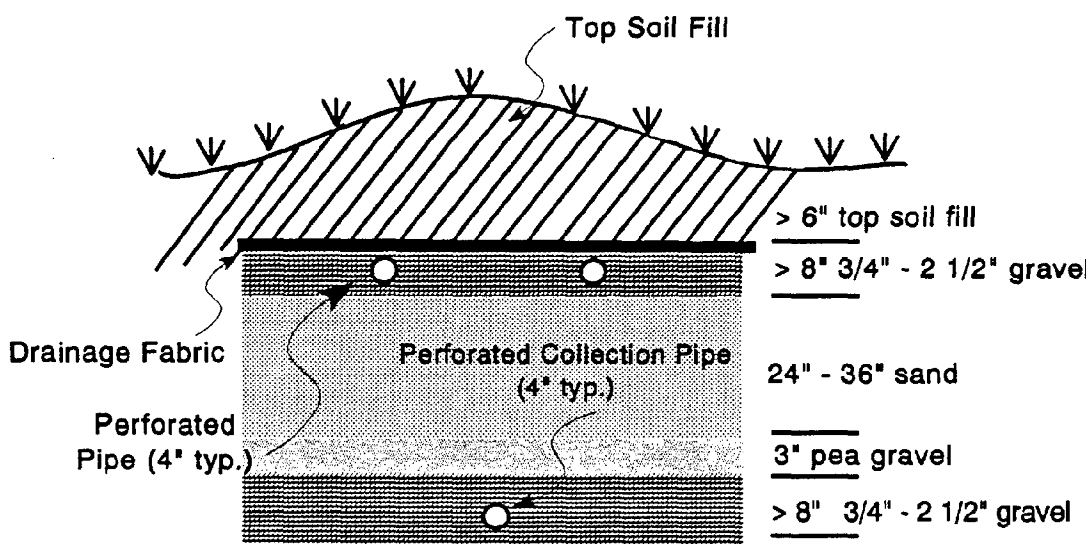
Free Access Filters

The plan and profile of a typical free access filter appear in Figure 2. These filters are often built within the natural soil, but may also be constructed completely above the ground surface. They are usually surrounded by sidewalls, often of masonry construction, to prevent earth from washing into the filter media and to confine the flow of wastewater. Where severe climates are encountered, filter walls should be insulated if exposed directly to the air. The floor of the filter is often constructed of poured concrete or other masonry, but may consist of the natural compacted soil. It is usually sloped to a slight grade so that effluent can be collected into open joint or perforated underdrains.

Figure 1. Typical Buried Intermittent Sand Filter System



Profile View

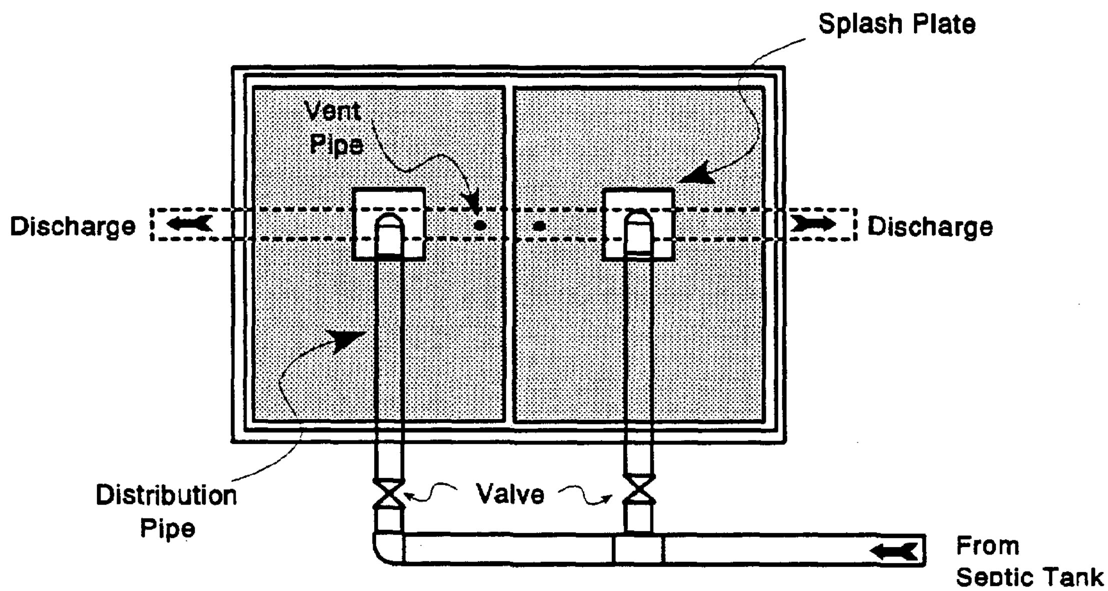


Section View

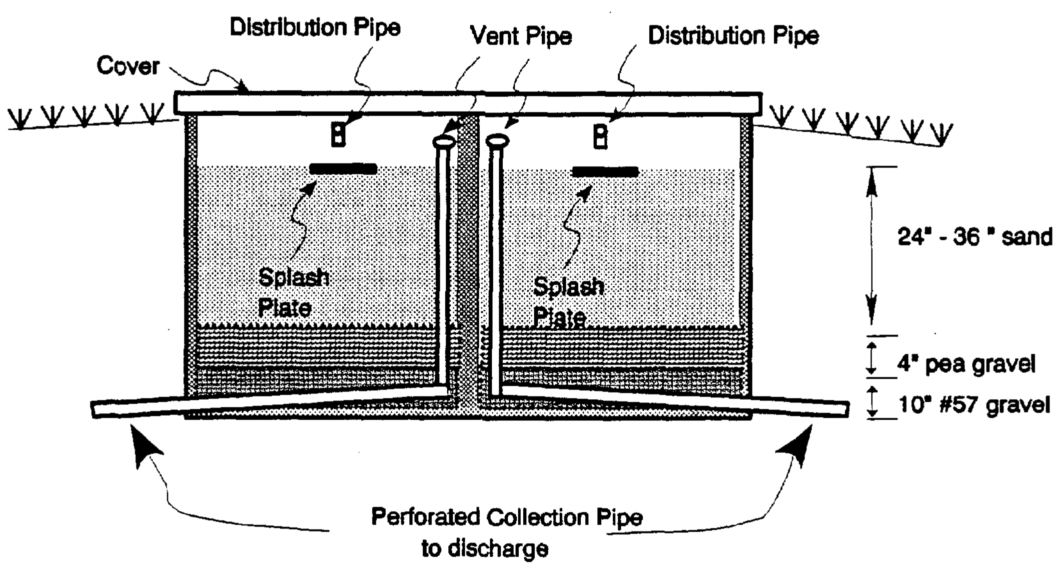
Not To Scale
Not All Details Are Shown

Technical Services

Figure 2. Free Access Intermittent Sand Filter Plan and Cross Section View



Plan View



Cross Section

Technical Services

Not To Scale
Not All Details Are Shown
(DDE/7-20-92)

Free access filters may be covered to protect against severe weather conditions, and to avoid encroachment of weeds or animals. The cover also serves to reduce odor conditions. Covers may be constructed of treated wooden planks, galvanized metal, or other suitable material. Screens or hardware cloth mounted on wooden frames may also serve to protect filter surfaces. Where weather conditions dictate, covers should be insulated cover and sand surface.

The underdrain lines should be constructed of an acceptable material with a minimum diameter of 4 in. (10 cm). The tile is normally laid so that joints are spaced not less than 1/4 in. (0.6 cm) or greater than 1/2 in. (1.3 cm) apart. Conventional perforated pipe may also be employed for distribution and collection. The underdrain lines may be laid directly on the filter floor, which should be slightly pitched to carry filtered effluent to the drain line. In shallow filters, the drain line may be laid within a shallow trench within the filter floor. Drain lines are normally spaced at 12-ft (3.6-m) centers and sloped at approximately 0.5 to 1% grade to discharge. The upstream end of each drain line should be vented with a vertical vent pipe above the filter surface, but within the covered space.

The bedding material for underdrain lines should be a minimum of 10 in. (25 cm) of washed graded gravel or stone with sizes ranging from 1/4 to 1-1/2 in. (0.6 to 3.8 cm). The gravel or stone may be overlain with a minimum of 3 in. (8 cm) of washed pea gravel interfacing with the filter media.

Distribution to the filter may be by means of troughs laid on the surface, pipelines discharging to splash plates located at the center or corners of the filter, or spray distributors. Care must be taken to insure that lines discharging directly to the filter surface do not erode the sand surface. The use of curbs around the splash plates or large stones placed around the periphery of the plates will reduce scour. A layer of washed pea gravel placed over the filter media may also be employed to avoid surface erosion. This practice will create maintenance difficulties; however, when it is time to rake or remove a portion of the media surface.

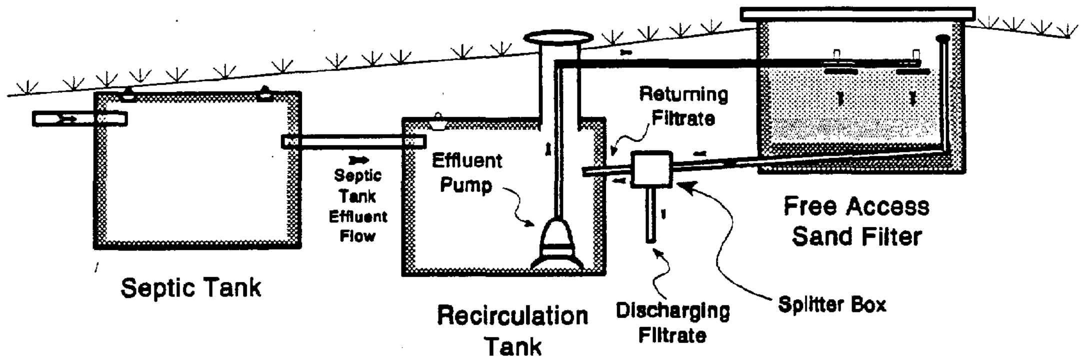
Filter media employed in free access filters may be any washed, durable granular material free of organic matter. As indicate previously for buried filters, mixtures of sand, slag, coal, or other materials may be employed, but with caution.

Recirculating Filters

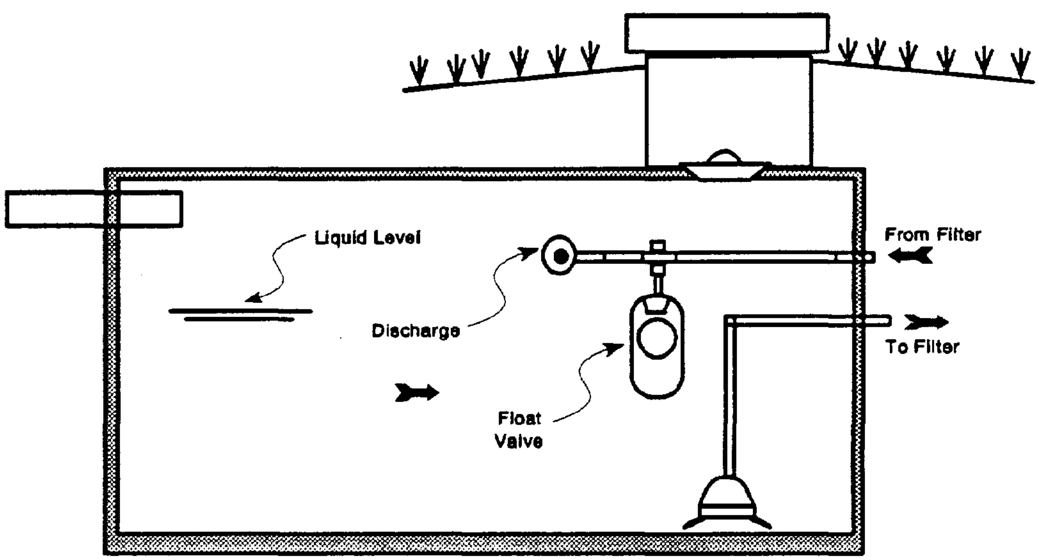
A profile of a typical recirculating sand filter system is presented in Figure 3. Recirculating filters are normally constructed with free access to the filter surface. The elements of filter construction are identical to those for the free access filter.

The basic difference between the recirculating filter and the free access filter is the recirculation chamber (dosing chamber) which incorporates a pump to recycle filter effluent. The recirculation tank receives the overflow from a septic tank, as well as a portion of sand filter effluent. A pump, controlled by a time clock mechanism, pumps the wastewater mixture to the filter surface. The recirculation tank is of equivalent strength and material to the septic tank. It is normally 1/4 to 1/2 the size of the septic tank (or a volume equivalent to at least one day's volume of raw wastewater flow). The tank must be accessible for maintenance of pumps, timers, and control valves. Covers should be provided and insulated as required by climatic conditions.

Figure 3. Typical Recirculating Sand Filter System (Using Splitter Box)



4. Cross-section Of A Recirculating Tank Using A Float Valve



Not To Scale
Not All Details Are Shown

(DDE/7-20-92)

Recirculation ratios may be controlled by a variety of methods. These include splitter boxes, moveable gates, check valves, and a unique "float valve" arrangement (Figure 4). The "float valve" incorporates a simple tee and a rubber ball suspended in a wire basket. The ball will float up and close off the inverted tee when the water level rises. Recirculation rations are normally established between 3:1 to 5:1.

Recirculation pumps are normally submersible pumps rated for 1/3 horsepower. They should be sized to empty the recirculation tank in less than 20 min. The recirculation pump should be controlled by a time clock to operate between 5 to 10 min every 30 min, and should be equipped with a float shut-off and high water override.

Operation and Maintenance

General

Intermittent sand filters require relatively little operational control or maintenance. Once wastewater is applied to the filter, it takes from a few days to two weeks before the sand has matured. BOD and SS concentrations in the effluent will normally drop rapidly after maturation. Depending upon media size, rate of application, and ambient develop. Winter start-up should be avoided since the biological growth on the filter media may not properly develop.

As discussed above, clogging of the filter eventually occurs as the pore space between the media grains begins to fill with inert and biological materials. Once hydraulic conductivity falls below the average hydraulic loading, permanent ponding occurs. Although effluent quality not initially suffer, anaerobic conditions within the filter result in further rapid clogging and a cessation of nitrification. Application of wastewater to the filter should be discontinued when continuous ponding occurs at levels in excess of 12 in. (30 cm) above the sand surface. A high water alarm located 12 in. (30 cm) above the sand surface serves to notify the owner of a ponded condition.

Since buried filters cannot be easily serviced, the media size is normally large and hydraulic application rates are low (usually less than 2 in./d [5 cm/d]). Proper pretreatment maintenance is of paramount importance. Free access filters, on the other hand, may be designed with finer media and at higher application rates. Experience indicates that intermittent sand filters receiving septic tank influent will clog in approximately 30 and 150 days for effective sizes of 0.2 mm and 0.6 mm, respectively. Aerobically treated effluent can be applied at the same rates for up to 12 months if suspended solids are under 50 mg/1. Results with recirculated filters using coarse media (1.0 - 1.5 mm) indicate filter runs in excess of one year.

Maintenance of Media

Maintenance of the media includes both routine maintenance procedures and media regeneration upon clogging. These procedures apply to free access filters only. The effectiveness of routine raking of the media surface has not been clearly established, although employed in several studies. Filters open to the air require weed removal as well. Cold weather maintenance of media may require different methods of wastewater application, including ridge and furrow and continuous flooding. These methods are designed to eliminate ice sheet development. Use of insulated covers permits trouble-free winter operation in areas with ambient temperatures as low as -40° F.

Eventually, filter clogging requires media regeneration. Raking of the surface will not in itself eliminate the need for more extensive rehabilitation. The removal of the top layer of sand, as well as replacement with clean sand when sand depths are depleted to less than 24 to 30 in. (61 to 76 cm), appears to be very effective for filters clogged primarily by a surface mat. This includes filters receiving aerobically treated effluent. In-depth clogging, however, often prevails in many intermittent filters requiring oxidation of the clogging materials. Resting of the media for a period of time has proven to be very effective in restoring filter hydraulic conductivity. Hydrogen peroxide treatment may also prove to be effective, although insufficient data are available on long-term application of this oxidizing agent.

Other Maintenance Requirements

The successful operation of filters is dependent on proper maintenance of the pretreatment processes. The accumulation of scum, grease, and solid materials on the filter surface due to inadequate pretreatment results in premature filter failure. This is especially critical for buried filters. Grease traps, septic tanks, and other pretreatment processes should be routinely maintained in accordance with requirements listed in other sections of this manual.

Dosing chambers, pumps, and siphons should receive periodic maintenance checks. If electronic sensing devices are employed to warn owners of filter ponding, these devices should also be periodically checked as well.

Maintenance Summary

The maintenance and operational requirements for buried, free access and recirculating filters are summarized in Tables 4, 5, and 6. Routine maintenance requirements have not been well documented for intermittent filtration onsite, but visits should be made four times per year check filters and their appurtenances. Based on meager data base, unskilled manpower requirements for buried filter systems would be less than 2 man days per year for examination of dosing chamber and appurtenances and septic tank. Free access filters may require from 2 to 4 days per year for media maintenance and replacement and examination of dosing chamber, septic tank, and appurtenances. Additional time would be required by analytical technicians for effluent quality analysis as required. Power requirements would be variable, depending upon the dosing method employed, but should be less than 0.1 kWh/day. The volume of waste media from intermittent filters may amount to approximately 0.25 ft^3/ft^2 (0.08 m^3/m^2) of surface area each time media must be removed.

Table 4. Operation And Maintenance Requirements For A Buried Intermittent Sand Filter

ITEM	O/M REQUIREMENTS
Pretreatment:	Depends upon process
Dosing Chamber:	
Pumps and controls	Check every 3 months
Timer sequence	Check and adjust every 3 months
Appurtenances	Check every 3 months
Filter Media:	None

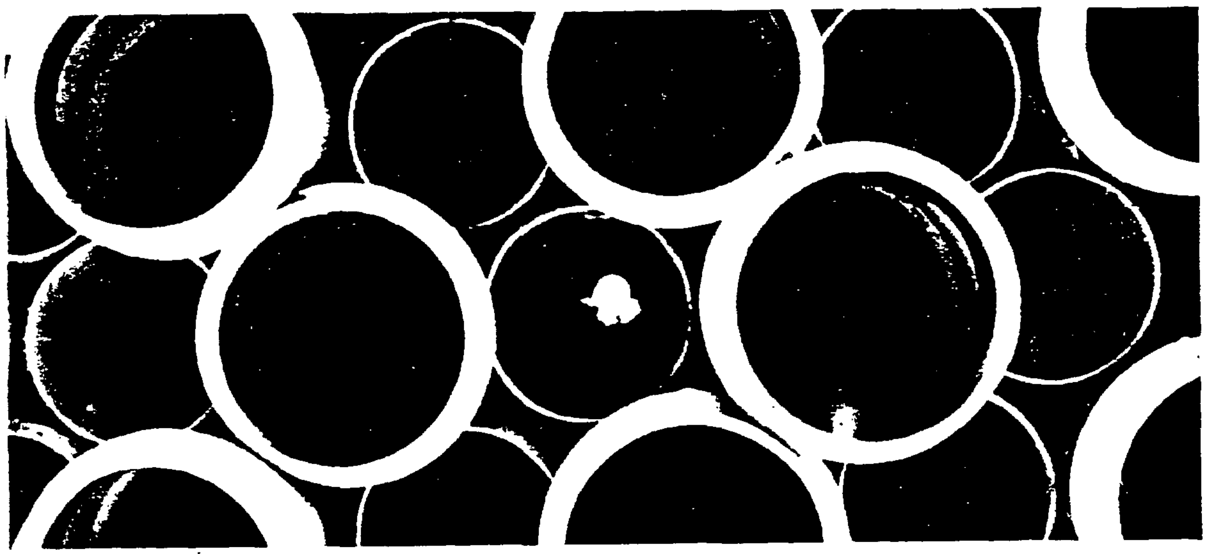
Table 5. Operation And Maintenance Requirements For A Free Access Intermittent Sand Filter

ITEM	O/M REQUIREMENTS
Pretreatment:	Depends upon process
Dosing Chamber:	
Pumps and controls	Check every 3 months
Timer sequence	Check and adjust every 3 months
Appurtenances	Check every 3 months
Filter Media:	
Raking	Every 3 months, 3 in. deep
Replacement	
-Septic Tank feed	Replace when ponded more than 12 in. deep; replace top 2 to 3 in. sand; rest while alternate unit in operation (60 days)
-Aerobic feed	Replace when ponded more than 12 in. deep; replace top 2 to 3 in. sand; return to service
Other:	Weed as required; maintain distribution device as required; protect against ice sheeting; check high water alarm

Table 6. Operation And Maintenance Requirements For Recirculating Intermittent Sand Filters

ITEM	O/M REQUIREMENTS
Pretreatment:	Depends upon process
Dosing Chamber:	
Pumps and controls	Check every 3 months
Timer sequence	Check and adjust every 3 months
Appurtenances	Check every 3 months
Filter Media:	
Raking	Every 3 months skim sand 3 in. deep when heavy incrustation occur;
Replacement	Add new sand when sand depth falls below 24 in.
Other:	Weed as required; maintain distribution device as required; protect against ice sheeting

Design and Installation of Low-Pressure Pipe Waste Treatment Systems



Craig Cogger Bobby L. Carlile Dennis Osborne Ed Holland

UNC Sea Grant College Publication UNC-SG-82-03
May 1982

Design and Installation of Low-Pressure Pipe Waste Treatment Systems

Written by Craig Cogger, Bobby L. Carlile and Dennis Osborne
North Carolina State University
Department of Soil Science

Ed Holland
Triangle J Council of Governments

Designed and edited by Kathy Hart
UNC Sea Grant College Program

Illustrated by Timothy Howard

Cover Photo by Neil Caudle

UNC Sea Grant College Publication UNC-SG-82-03
May 1982

Introduction

Many sites under consideration for development in North Carolina are not suitable for on-site sewage disposal by conventional septic systems. Among these sites are some which do have enough depth and area of usable soil to provide safe disposal via low-pressure pipe (LPP) systems. LPP systems are not a panacea for all the unsuitable soils of North Carolina, but they are useful for some specific conditions where conventional systems have frequently failed.

This manual specifies the procedures and materials to be used for successful siting, design, installation and maintenance of residential LPP systems. Use of proper materials and techniques is critical to the success of the LPP, as well as to all other ground absorption systems. Many engineers, sanitarians, contractors and designers are unfamiliar with LPP construction, and these instructions are designed as an aid to them. Although those who design, build and use septic systems can benefit from this report, it must always be used in cooperation with the local health department. The local health department must first approve a site, and then assign waste flow and soil loading rates.

This manual covers design and installation of small LPP systems suitable for homes and small businesses. Principles are similar for larger commercial and institutional systems, but the special requirements of those systems are not addressed.

CHAPTER 1

What Is Low-Pressure Pipe Distribution?

A soil-absorption system must serve two purposes: 1) keep untreated effluent below the surface, and 2) purify the effluent before it reaches ground or surface water. The system works best when the distribution area is not saturated with water or effluent, allowing efficient aerobic bacteria to treat the wastes.

There are several conditions which frequently hinder the operation of soil-absorption systems. Clogging of the soil can occur from localized overloading during use or from the mechanical sealing of the soil-trench interface during construction. This clogging can cause effluent to break through to the surface, especially in fine-textured soils. Anaerobic conditions caused by continuous saturation due to overloading or a high-water table retard treatment, increasing the potential for pollution. Shallow soils are not deep enough to purify the effluent.

The LPP system has three design improvements to help overcome these problems. These are:

- uniform distribution of effluent
- dosing and resting cycles
- shallow placement of trenches

Problems from local overloading are decreased when effluent is distributed over the entire absorption area. Dosing and resting cycles help maintain aerobic conditions in the soil, improving treatment. Shallow placement increases the vertical separation from the system to any restrictive horizon or seasonally high-water table.

An LPP system is a shallow, pressure-dosed soil-absorption system (Figure 1). It consists of:

- two-compartment septic tank
- pumping chamber
- submersible effluent pump and level controls
- high-water alarm
- supply line and manifold
- distribution laterals
- suitable area and depth of soil

When septic tank effluent rises to the level of the upper pump control, the pump turns on and effluent moves through the supply line and distribution laterals. These laterals are PVC pipes containing small holes (⅛ inch to ¼ inch) spaced three to five feet apart. The pipes are placed in narrow trenches six to 18 inches deep, spaced five or more feet apart. Under low pressure [0.7 to two pounds per square inch (psi)] supplied by the pump, septic tank effluent flows through the holes and into the trenches. It diffuses from the trenches into the soil where it is treated.

The pump turns off when the effluent level falls to the lower control. The level controls are set so that the effluent is pumped two to four times daily with resting periods in between to allow aerobic treatment of effluent. If the pump or level controls should fail, the effluent would rise to the level of the alarm control. The alarm would turn on, signaling the homeowner of failure.

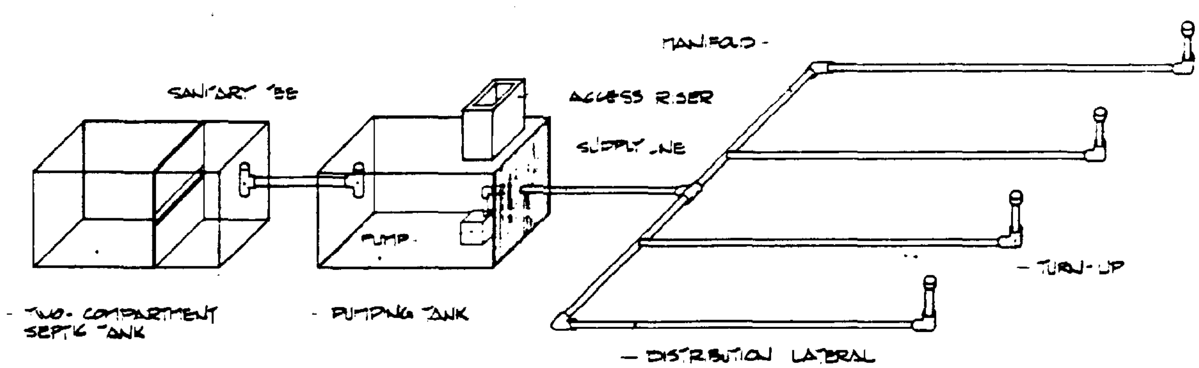


Figure 1. Basic components of a low pressure pipe system

CHAPTER 2

Site and Soil Requirements for LPP Systems

The suitability of an LPP system for a given site is determined by the soil, slope and available space, as well as by the anticipated waste flow. The criteria below are a set of practical guidelines that may be modified by individual county health departments.

Space requirements

The distribution network of most residential LPP systems occupies from 1000 square feet to 5000 square feet of area depending on the soil permeability and design waste load. In addition, an area of equal size must be set aside for future repair or replacement of the system. Space between the existing lateral lines is not a suitable repair area, unless the initial spacing between lines is 10 feet or wider. The septic tank, pumping chamber, distribution field and repair area are also all subject to horizontal setbacks from wells, property lines, building foundations, etc., as specified in local or state regulations [10 NCAC 10A .1912(a)]. Although it is not feasible to integrate all of the site and soil setback criteria into a general lot size requirement, an undeveloped lot smaller than one acre will not usually be acceptable for an LPP system.

Soil requirements

An LPP system should be situated on the best soil and site on the lot. A minimum of 12 inches of usable soil is required between the bottom of the absorption field trenches and any underlying restrictive horizons such as consolidated bedrock or hardpan, or to the seasonally high, water table. LPP trenches can be placed as shallowly as eight to 12 inches deep, giving a minimum soil-depth requirement of 20 to 24 inches. The soil must be of suitable or provisionally suitable texture, structure and permeability, as defined in state regulations (10 NCAC 10A .1920). In some cases where the depth to the seasonal water table or restrictive horizons is less, a modified LPP may be installed using imported fill. Great care must be used in building these systems. Their design and construction are covered in Chapter 8.

Topography

Low-pressure distribution fields located on slopes require special design and installation procedures (Chapter 7). The distribution field of any LPP system should be at an elevation equal to or higher than the pumping chamber. This prevents the gravity flow or inadvertent siphoning of effluent from the pump chamber to the field when the pump is not operating. If the field must be lower than the pump tank, then the system must be designed to ensure that effluent cannot leave the pump chamber when the pump is turned off.

Drainage requirements

Depressions, gullies, drains and erosional areas must be avoided to prevent hydraulic overloading by surface runoff. Neither the septic tank, pumping chamber nor distribution field should be located in such areas. Surface water and perched groundwater must be intercepted or diverted away from all components of the LPP system.

CHAPTER 3

Layout of an LPP System

The next three chapters are a step-by-step procedure for designing an LPP system. There is no one LPP that fits all sites—each must be designed individually. Additional procedures used when designing LPP systems on sloping sites and where fill is used are covered in Chapters 7 and 8.

Size of the absorption area

The total amount of absorption area depends on two factors—the daily wastewater flow of the system and the absorptive capacity of the soil.

Step 1. Calculate daily waste flow. For residential systems, the estimated flow is 150 gallons per day (gpd) for each bedroom (BR) in the house.

Example:

For a 3-BR house:
Flow = 150 gpd/BR x 3 BR = 450 gal

Step 2. Determine the loading rate. Estimate soil permeability during the field evaluation and determine the wastewater-loading rate using Table 1.

Example:

For a sandy clay loam:
Loading rate = 0.25 gpd/ft^2
Note: Waste flow and loading rates must be determined by the local health department before the LPP system can be designed.

Step 3. Compute the total area needed for the absorption system using the equation: Area = flow/loading rate.

Example:

Using flow and loading rates calculated above:
Area = 450 gpd/0.25 gpd/ft^2 = 1800 ft^2

Step 4. Determine total length of distribution lines. Spacing between lines must be five feet or more to prevent overloading. Divide total area by five to obtain the total length of the distribution lines.

Example:

Length = 1800 ft^2/5 ft = 360 ft

Table 1. Maximum loading rates for LPP systems based on soil texture and estimated permeability

USDA Soil Texture*	Estimated Permeability	Maximum Loading Rate**
	min/in.	*gpd/ft^2*
Sand, loamy sand	20	0.50-0.40
Sandy loam, silt loam	20-40	0.40-0.30
Sandy clay loam, clay loam	40-60	0.30-0.20
Silty clay loam, sandy clay	60-90	0.20-0.10
Silty clay, clay	90-120	0.10-0.05

*This table does not consider the effects of clay mineralogy on soil permeability. A sandy clay composed of 1:1 clays may be more permeable than a clay loam of 2:1 clays.

**These loading rates should be used only for calculating the size of LPP systems—not for other types of systems.

Step 5. Calculate gravel requirements. To fill a six-inch wide trench six inches deep with gravel, 1.5 yards (1.9 tons) is needed per 100 feet of line.

Example:

For 360 ft of line:
Gravel needed = (360 ft/100 ft) x 1.5 yds
= 5 yds

Size of septic and pumping tanks

Septic-tank volume is determined according to state and local regulations, and is the same as a conventional system. The pumping tank should provide one day for emergency storage; thus, it should be at least twice the volume (V) of the daily waste flow.

Example:

For a 450 gpd waste flow:
V pumping tank = 450 gal x 2 = 900 gal

Location of system

The LPP should be located in the best available soil on the lot. All setback requirements from wells, lot lines and waterways must be observed. The exact location of the tanks as well as drainage and landscaping improvements must be noted. A repair or replacement space on suitable soil equal in area to the absorption field must be located.

Shape of absorption field

When selecting the best shape to fit in the desired location, lines must be placed on the contour. Also, lines should not extend more than 70 feet from the manifold (supply line) due to excessive friction loss. When using larger lateral lines, the manifold must be placed in the center of the distribution system rather than along the side (Figure 2). For a layout example, see Figure 3.

Landscaping and drainage

All landscaping, filling and site drainage to be done before and after the LPP installation must be recorded in detail on the improvements permit.

Depth of lines

Lines are normally placed 18 inches deep. Shallower placement will be necessary in soils with shallow water tables, bedrock or restrictive horizons in order to meet the one-foot vertical separation requirement.

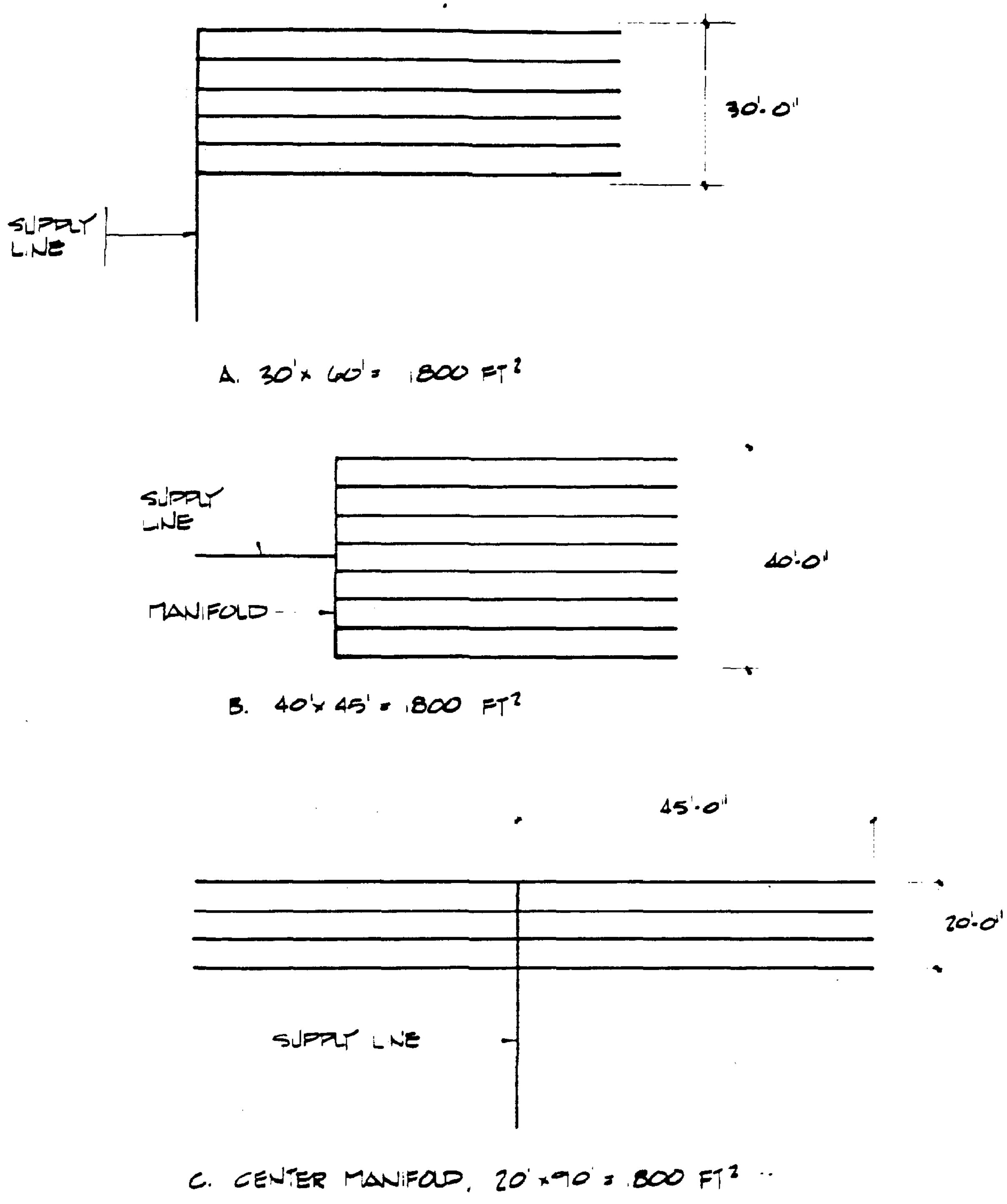
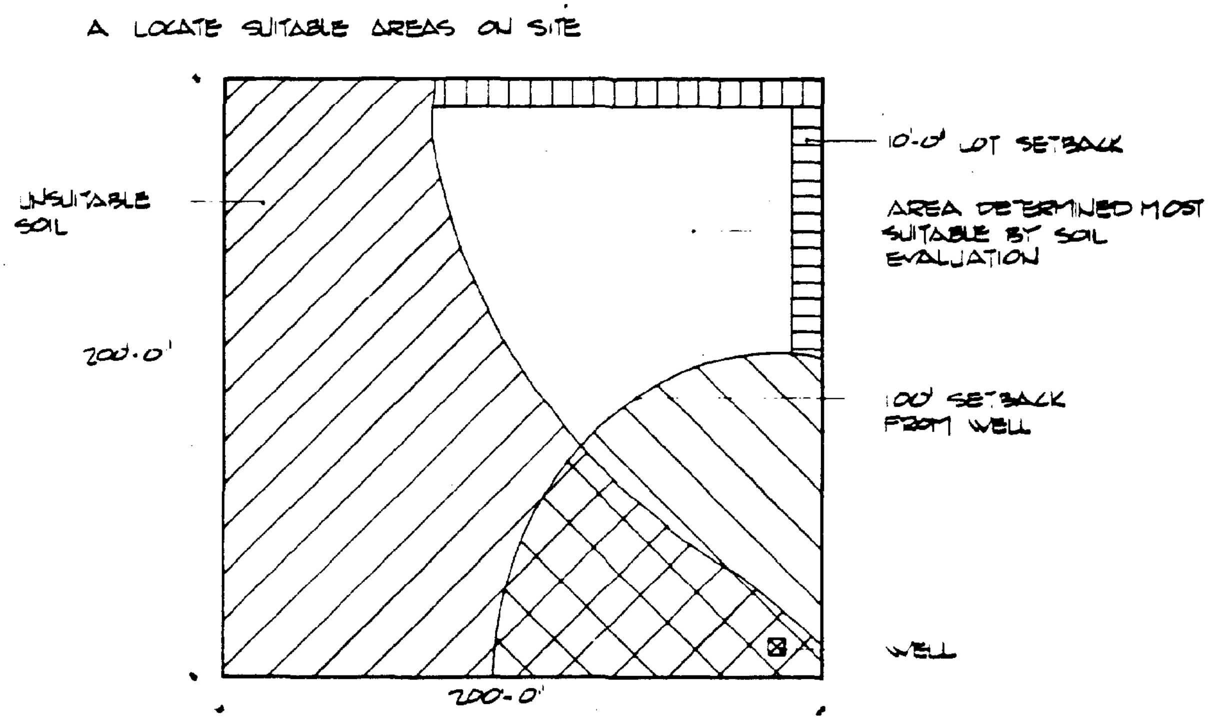


Figure 2. Three possible shapes of an 1800 ft² LPP distribution field



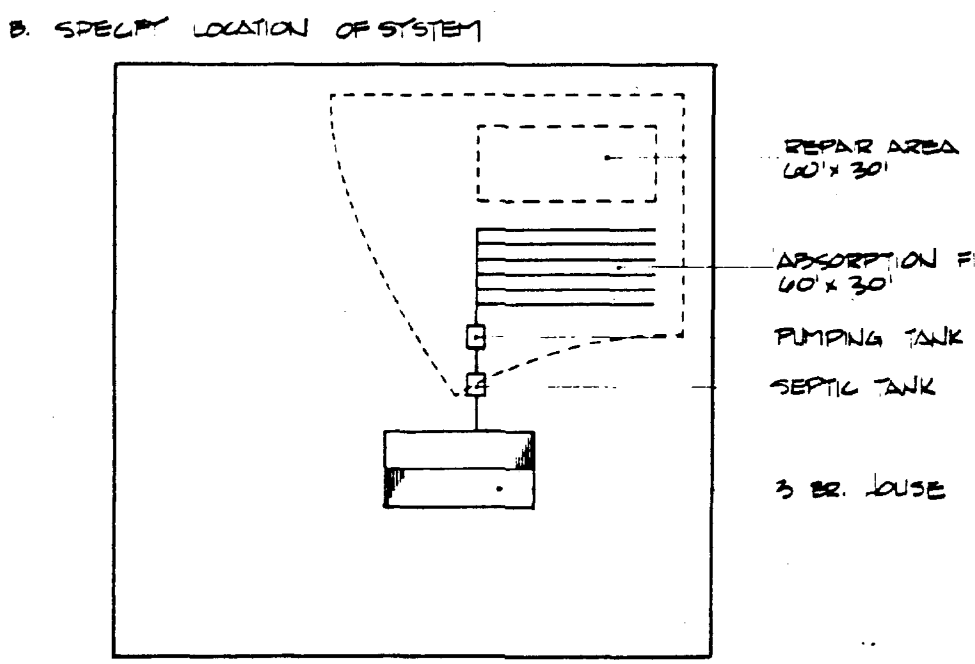


Figure 3. Layout of a sample system

CHAPTER 4

Dosing and Distribution System Design

The purpose of low-pressure dosing is to provide uniform distribution of septic tank effluent over the entire soil-absorption system. This is best achieved at a pressure head of two to four feet (0.9 to 1.7 psi). Lower pressures do not provide uniform delivery of effluent. Higher pressures cause local scouring of the gravel and soil in the trench bottoms. The proper dosing involves balancing the size of the distribution system with the dosing volume, pumping capacity, desired pressure and flow rate.

Dosing rate

The dosing rate depends on the pressure head and the size and number of holes in the distribution lines. Pressure head can range from two to four feet for adequate performance; holes must be ⅛ inch or greater in diameter, and hoi spacing can range from three to five feet. On s ping lots, it may be necessary to have holes as small as 3/32 inch and spacing greater than five feet, in a part, but not in all of the system as explained in Chapter 8. The best starting values for calculation are a 5/32-inch hole diameter, five-foot hole spacing and three feet of pressure head.

Step. 1. Calculate the number (no.) of holes.
No. holes = length of line/hole spacing

Example:

For a system with 5-ft hole spacing and six 60-ft lines:
No. holes/line = 60 ft/5 ft/hole
= 12 holes/line
Total holes = 12 holes/line x 6 lines
= 72 holes

Step 2. Determine the flow rate per hole. This is calculated from the hole size and pressure head using Table 2.

Example:

For 3-ft pressure head and 5/32-inch holes:
Flow rate = 0.50 gallons per minute (gpm)

Step 3. Calculate total dosing rate.

Example:

Flow rate/hole = 0.50 gpm
Flow rate/line = 0.50 x 12 holes = 6.0 gpm
Total flow rate = 0.50 x 72 holes = 36 gpm

For systems where the absorption field is at a lower elevation than the pump, a ¼-inch siphon-breaker hole must be drilled in the supply line in

Table 2. Flow rate as a function of pressure head and hole diameter in drilled PVC pipe

Pressure Head		Hole diameter (in.) 3/32	1/8	5/32	3/16	7/32
ft	*psi*			*– Flow rate (gpm) –*		
1	0.43	0.10	0.18	0.29	0.42	0.56
2	0.87	0.15	0.26	0.41	0.59	0.80
3	1.30	0.18	0.32	0.50	0.72	0.98
4	1.73	0.21	0.37	0.58	0.83	1.13
5	2.16	0.23	0.41	0.64	0.94	1.26

the pumping tank. This hole will prevent inadvertent siphoning of the contents of the pump tank into the field. An extra two gallons per minute must be added to the pumping rate to compensate for flow through the siphon-breaker hole.

Example:

For a system with 36 gpm flow rate and a siphon-breaker hole.
Total flow rate = 36 gpm + 2 gpm = 38 gpm

Pump selection

The pump must have enough power to pump effluent at the calculated flow rate against the total head (resistance) encountered in the distribution system. The total head is the amount of work the pump must do to overcome elevation (gravity) and friction in the system at the specified pressure and flow rate. Total head = elevation head + pressure head + friction head.

Elevation head is the difference in elevation from the pump to the end of the manifold. Remember that the pump will be four feet or five feet below ground level in the pumping chamber.

Pressure head is the pressure required for even distribution and is usually specified between two and four feet.

Friction head is the loss of pressure due to friction as the effluent moves down the pipes. Pipe friction is estimated using Table 3. When estimating pipe friction, use the length of the supply manifold, but not the lateral lines. Add 20 percent to the pipe friction estimate to account for friction loss in joints and fittings. Note that friction loss varies with pumping rate as well as with pipe length and diameter.

The total head must be calculated to select the proper size pump.

Step 1. Compute friction head.
Friction head = 1.2(pipe friction)

Table 3. Friction loss per 100 feet of PVC pipe

	Pipe diameter (in.)					
Flow	1	1¼	1½	2	3	4
gpm	*Friction loss (ft)*					
1	0.07					
2	0.28	0.07				
3	0.60	0.16	0.07			
4	1.01	0.25	0.12			
5	1.52	0.39	0.18			
6	2.14	0.55	0.25	0.07		
7	2.89	0.76	0.36	0.10		
8	3.63	0.97	0.46	0.14		
9	4.57	1.21	0.58	0.17		
10	5.50	1.46	0.70	0.21		
11		1.77	0.84	0.25		
12		2.09	1.01	0.30		
13		2.42	1.17	0.35		
14		2.74	1.33	0.39		
15		3.06	1.45	0.44	0.07	
16		3.49	1.65	0.50	0.08	
17		3.93	1.86	0.56	0.09	
18		4.37	2.07	0.62	0.10	
19		4.81	2.28	0.68	0.11	
20		5.23	2.46	0.74	0.12	
25			3.75	1.10	0.16	
30			5.22	1.54	0.23	
35				2.05	0.30	0.07
40				2.62	0.39	0.09
45				3.27	0.48	0.12
50				3.98	0.58	0.16
60					0.81	0.21
70					1.08	0.28
80					1.38	0.37
90					1.73	0.46
100					2.09	0.55

Example:

For a 70-ft supply line with a 2-in. diameter and a 36-gpm pumping rate:
Pipe friction = (70 ft/100 ft) x 2.2 ft
= 1.5 ft
Friction head = 1.2 x 1.5 ft
= 1.8 ft

Step 2. Calculate total head.

Example:

For a system with 5-ft elevation head from pump to end of the lines, 3-ft pressure head, 1.8-ft friction head:
Total head = 5 ft + 3 ft + 1.8 ft
= 9.8 ft

The system will require a pump with a capacity of 36 gallons per minute against 10 feet of head. It is always necessary to specify the total head when selecting a pump. The head and flow requirements are checked against the performance curve provided by the pump manufacturer. Examples of performance curves are shown in Figure 5. It is important to use the performance curve for the specific brand and size of pump to be used. Performance curves vary among brands.

Step 3. Select a pump of proper capacity. Consult the appropriate performance curve. The system requirements of flow and total head (36 gallons per minute at 10 feet) intersect at a point which must fall below the performance curve. If the point falls above the curve, then the pump is too small.

Example:

This point in Figure 4 falls below the curve; therefore, the pump is adequate.

When the chosen pump is too small, there are several options to consider:

- Select a larger pump.
- Reduce the total head requirement by reducing the pressure head (two feet is the minimum). This has a large effect, as a lower pressure head will also lower the flow rate and friction head.
- Reduce the friction-head loss by using a larger diameter supply manifold (two inches is a practical maximum for residential systems).
- Reduce the flow rate by using a smaller hole size (⅛-inch is the minimum) or by increasing hole spacing.
- Raise the pump by placing more blocks underneath it.

A combination of choices can be made. The goal is to design a system that works properly for the lowest possible price. A larger pump is an easy solution, but will be more expensive than one of the other options. For most residential systems a 0.3- to 0.4-horsepower pump will be adequate with judicious selection of the other parameters.

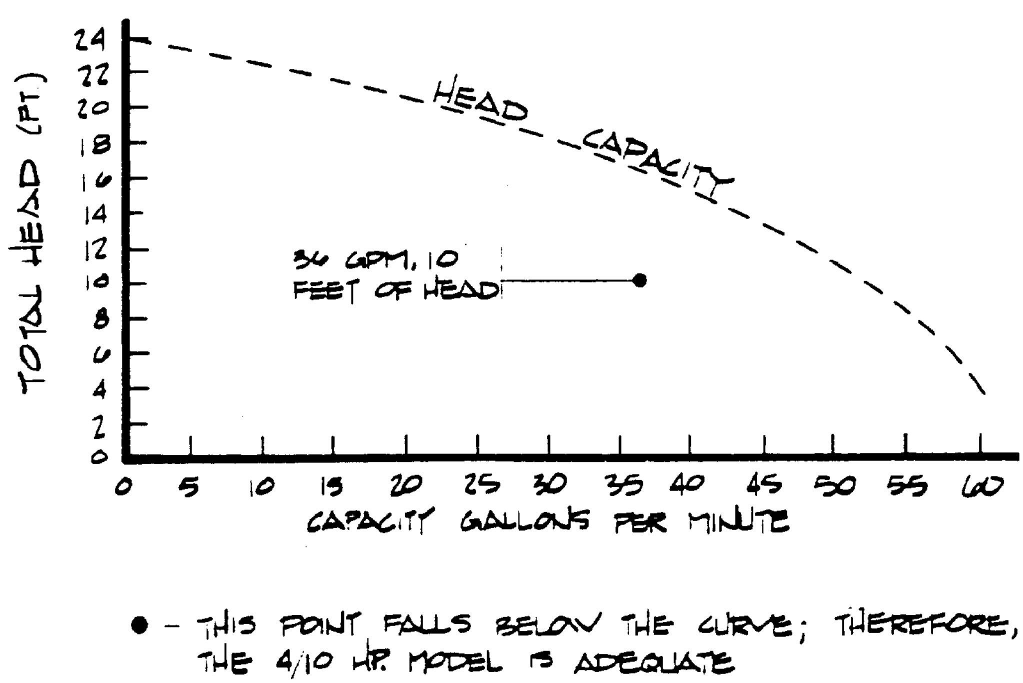


Figure 4. Comparing pumping requirements to performance curves

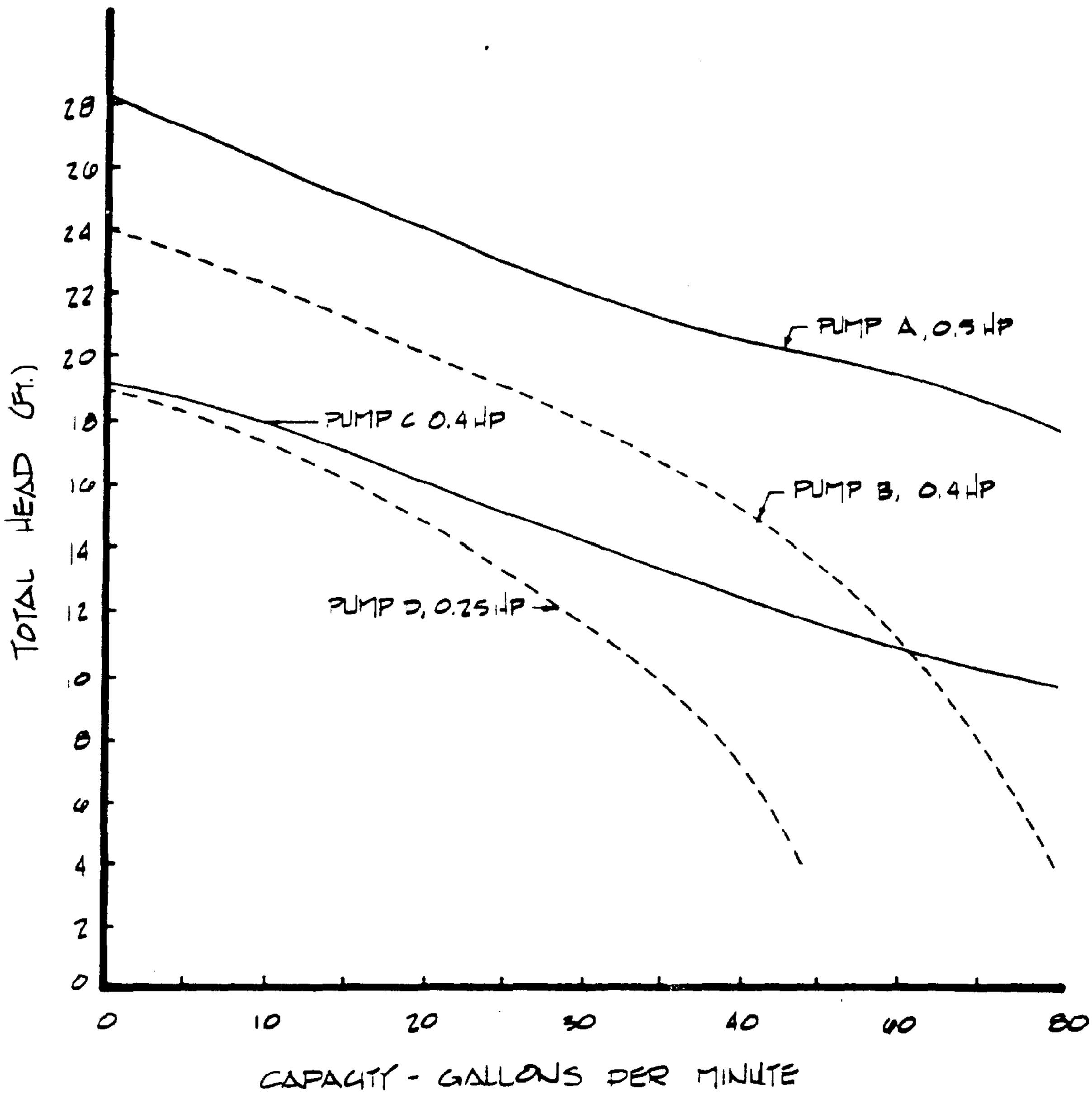


Figure 5. Examples of performance curves (capacity vs total head) for four submersible effluent pumps

Dosing volume

Dosing volume is the amount of effluent pumped to the absorption field each time the pump runs. The dosing volume must be large enough to provide adequate distribution in the field and adequate resting time between doses, yet small enough to avoid overloading. The minimum dose to provide adequate distribution depends on the size of the supply and lateral network.

Step 1. Calculate minimum dosing volume.
V dose = V supply + 5 (V laterals)
The minimum volume is the sum of the supply-line volume and five times the volume of the lateral lines. The volume of the lines is calculated using Table 4.

Table 4. Storage capacity per 100 ft of PCV pipe

	Storage Capacity	
Pipe Diameter	160 psi	Schedule 40
in.	*gal/100 ft*	
1	5.8	4.1
1¼	9.0	6.4
1½	12.5	9.2
2	19.4	16.2
3	42.0	36.7

Example:

1. Supply line = 70 ft of 2-in. pipe
   V supply = (70/100) x 19.4 gal
   = 13.6 gal
2. Lateral lines = 360 ft of 1¼-in. pipe
   V lateral = (360/100) x 9.0 gal
   = 32.4 gal
3. V dosing = 13.6 gal + 5 (32.4 gal)
   = 176 gal

Dosing two to four times per day provides adequate resting time. For a 450 gallon-per-day design, this would be a range of 112 to 225 gallons per dose (gal/dose).

Step 2. Select dosing volume.

Example:

Selecting 180 gal/dose would give between two and three doses per day. This volume is larger than the minimum in Step 1. If water use is less than 450 gpd, dosing will occur less frequently, providing longer resting periods between doses.

Step 3. Compute the depth of effluent pumped per dose. In order to set the pump controls to deliver the proper dose, the depth of effluent to be pumped from the tank for each dose must be calculated. The computation is done using the following equation: Dosing depth = (V dose/V tank) x liquid depth of tank.

Example:

For a 900-gal pumping tank, 4-ft liquid depth (bottom of tank to outlet tee); 180-gal dose:
Dosing depth = (180 gal/900 gal) x 4 ft
= 0.8 ft = 9.6 in.

The float control switch for the pump should be set for a 10-inch drawdown to provide automatic doses of 180 gallons.

Check-valve calculation

Any effluent which remains in the supply and lateral lines of a properly sited system will drain back to the pumping chamber when the pump shuts off. If this volume is too large, it can cause overuse of the pump and excessive consumption of electricity. A check valve may be needed to prevent this return flow to the pumping chamber, especially on a large system with a long pumping distance. Check valves should be avoided if possible because they may malfunction when used for septic tank effluent. In general, a check valve should only be used if the total storage volume of the pipes is greater than one fourth of the total daily waste flow.

Step 1. Calculate storage volume.
V storage = V supply + V laterals

Example:

V storage = 13.6 gal + 32.4 gal
= 46.0 gal

Step 2. Compare to ¼ daily waste flow.

Example:

450 gpd x ¼ = 112 gal
46.0 gal < 112 gal
No check valve needed.

CHAPTER 5

Equipment Specifications

All necessary equipment and tools should be clearly listed so they can be obtained prior to building an LPP. To prepare this list, first consolidate the design specifications onto a single worksheet (Appendix 1). A copy of this worksheet along with an accurate sketch including drainage and landscaping requirements (Figure 3) should be filed for every system which is installed. Using this sheet, prepare a list of materials (Appendix 2). Be sure that the materials meet the requirements discussed below. A sketch of the distribution lines (Figure 6) and the pump system (Figure 7) are useful for counting the fittings.

Septic tank and pumping chamber

As noted earlier, an LPP system has two separate tanks—a septic tank and a pumping chamber. If a conventional septic system is being replaced by an LPP, the existing septic tank can be used (after being pumped out), and only one additional tank installed.

The septic tank receives wastewater directly from the house. It is sized according to state and local regulations for conventional systems (10 NCAC 10A .1907). The septic tank must be of two-compartment design for maximum solids retention. It is very important that the septic tank and pumping chamber are watertight. One-piece tanks are best. When using two-piece tanks, the tongue-in-groove joint must be carefully sealed with asphalt rope mastic.

Effluent from the two-compartment septic tank flows by gravity through a four-inch solid PVC pipe to the pumping chamber. The pumping chamber should have a liquid capacity of at least two times the daily wasteflow from the house, and can be a single-compartment design.

Both the septic tank and pumping chamber must be provided with aboveground concrete or masonry (or their equivalent) manhole risers to provide easy access for clean-out and pump service. The riser should be placed over the primary chamber of the septic tank and above the pump access hole in the pumping chamber. Risers should be wide enough to accommodate the existing lids on the tanks, should extend at least six inches above the finished grade of the site and should also be covered with a concrete lid. Standard well tiles can be used for the risers, provided that the inside diameter is larger than the access hole in the tank. All joints must be sealed to prevent the infiltration of surface runoff and groundwater to the tanks.

Pipe and fittings

All pipes and fittings in an LPP system should be made of PVC plastic. PVC is lightweight, easy to use and resists corrosion. All joints must be sealed with an appropriate PVC-solvent cement. The supply manifold from the pumping chamber to the LPP distribution field is usually 1½-inch or two-inch PVC, depending on specifications of the system (Chapter 4). A bushing or reducer may be needed to adapt the pump to the supply manifold. There should always be a threaded PVC union above the pump to allow easy removal or replacement. Lateral lines are usually made of 1¼-inch PVC. Appropriate holes in the laterals are drilled on site (Chapter 6).

PVC pipe may be of thin-wall (160 psi) or Schedule 40 specifications, but must be of the straight length variety. Thin-wall (160 psi) PVC is usually cheaper than Schedule 40. A globe or gate valve for final pressure adjustment is installed in the supply manifold inside the pumping chamber. The valve should be made of PVC or bronze, whichever is cheaper. All other tees, elbows, caps and reducers in the distribution system should be made of PVC. The end of each lateral line is equipped with a capped "turn-up" that provides aboveground access for clean-out or back-flushing (Figure 5). Using 45-degree elbows rather than 90-degree elbows for the turn-ups will make clean-out easier to do. Galvanized caps may be used if PVC is not available.

In the few instances where a check valve is necessary (Chapter 4), it should also be installed with threaded fittings in the pump chamber to provide easy access for maintenance.

Pump, float controls and alarm system

A good-quality, submersible effluent pump must be used in LPP systems. An expensive grinder pump is not required because the septic tank effluent will be relatively free of solid material. A septic-tank effluent pump or a submersible,

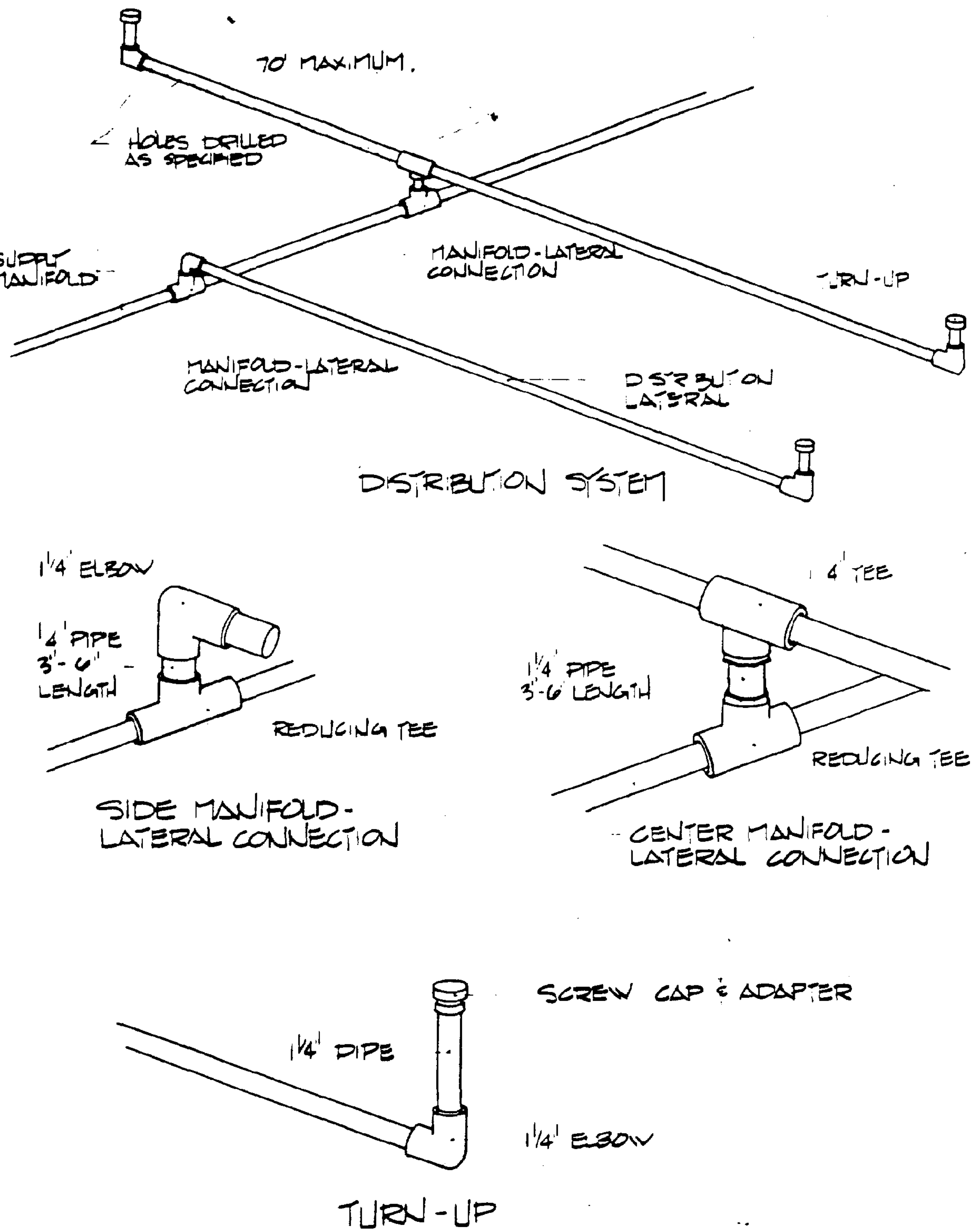


Figure 6. Details of distribution system

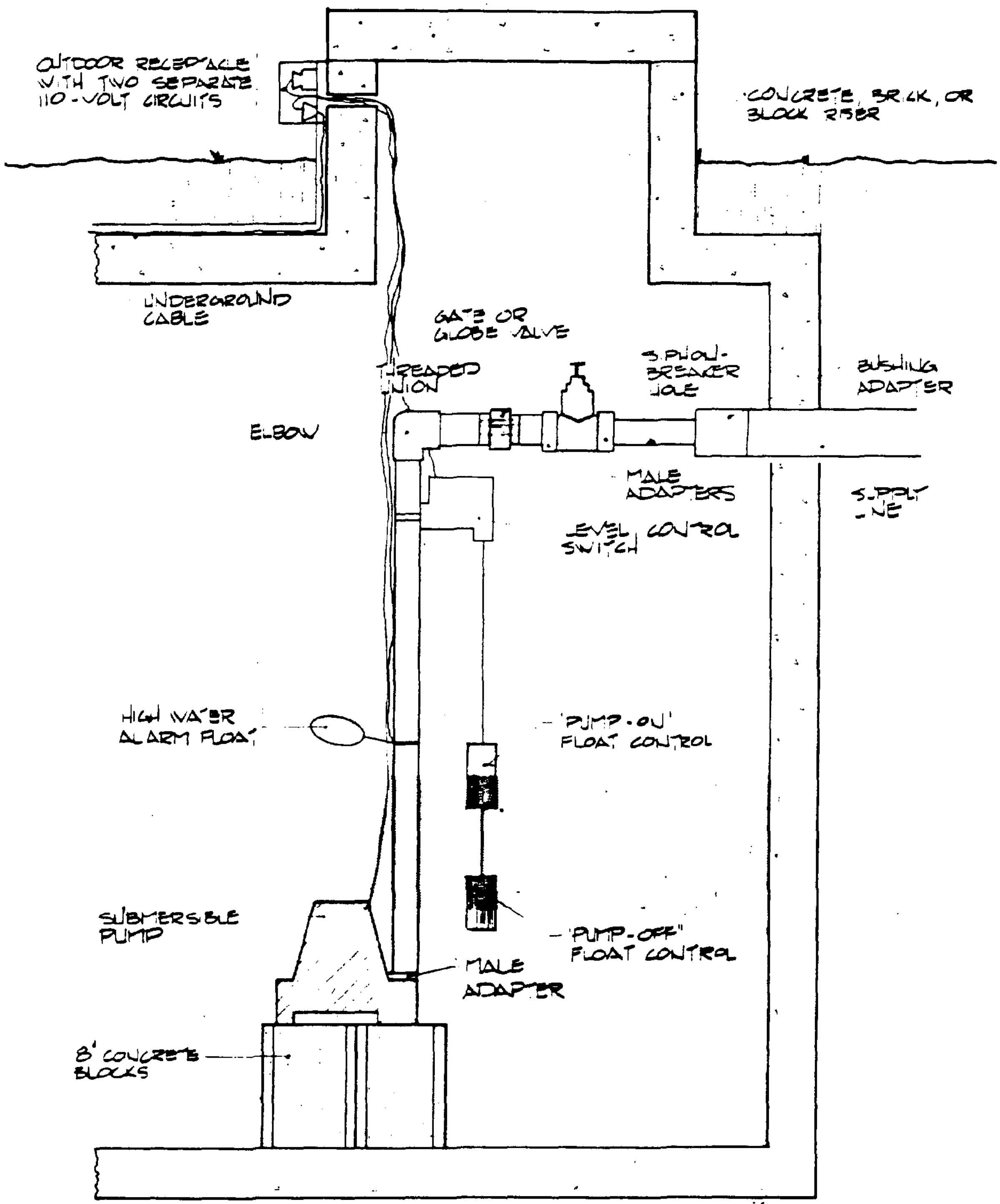


Figure 7. Details of pumping chamber

sump pump that will not be corroded by sewage should be used in the pumping chamber. Pumps with built-in switches should be avoided, unless the switch can be adjusted for the quantity of water to be pumped. The selection of pump size is discussed in Chapter 4. Pumps in the range of ¼ horsepower to 4/10 horsepower generally provide sufficient capacity for residential LPP systems, but the pumping requirements for each system must be checked against the performance curve of the pump to be used. It is better to use a slightly larger pump than necessary, because the final pressure can be adjusted with the in-line gate or globe valve.

The controls for the pumping system include a switching control for turning the pump on and off and a high-water alarm to signal pump malfunctions (Figure 6). The pump control system must be adjustable to meet the recommended loading rates for different sizes and shapes of pumping chambers. The controls must also be sealed against entry of corrosive and explosive gases from the effluent and should have NEMA (National Electrical Manufacturing Association) approval.

The two types of switches which have proven the most useful are magnetic, level-control switches and sealed, mercury switches. The magnetic level control consists of two floats suspended from a sealed, magnetic switch. This switch has been reliable and the pumping volume is easily adjusted. Mercury switches are activated by a sealed float which contains a tube of mercury in contact with power leads. Best performance has been obtained using two switches—one to close the pump circuit and the other to open it. Automatic timers with backup mercury floats have been successful in a few systems where uniform timing of the doses was important. Diaphragm and some mechanical float switches have not been acceptable for LPP use. The range of adjustment is often inadequate and the switches do not provide good service in a sewage environment.

In addition to the on and off control floats, there must be a separate float control for the high-water alarm. This may be a sealed, mercury-float switch mounted several inches above the on float. The high-water alarm should consist of a light bulb and/or audible signal mounted over a sign marked "wastewater system alarm" in a visible place in the home, such as the kitchen or utility room. It should be on a separate electrical circuit from the pump power line, and be equipped with a test switch. The alarm is activated if the water level in the pumping tank rises above the "pump-on" float control. The tank provides at least one day or more of excess storage capacity (depending on water use in the home) during which time the system must be repaired. Refer to Chapter 9 for repair and maintenance tips.

Complete control boxes for high-water alarms are available commercially. Simpler and cheaper systems can be assembled by an electrician. There are two basic requirements for an alarm system:

- It must operate on a separate electrical circuit from the pump.
- It must activate a labeled and easily visible (or audible) signal in the home whenever the water exceeds the normal "pump-on" level in the tank.

Gravel

LPP systems require about six inches of gravel in the lateral distribution trenches. Gravel size should be from ⅜ inch to one inch. Pea gravel or crushed rock may be used, but it must be washed. Gravel placement is discussed in Chapter 6.

Home water-saving devices

Any home with an LPP system must be equipped with l v-flow showerheads (three gallons per minut and low-flush commodes (3½ gallons or less per flush) in order to minimize the hydraulic load on the system. Those devices are a simple, low-cost way of reducing water consumption with no inconvenience to the homeowner. They are required by the North Carolina Building Code in all new construction. Low-flow showerheads and retro-fit dams for commode tanks should be used in any existing home where an LPP is installed.

CHAPTER 6

Installation Procedures

The actual installation of an LPP is simple and straightforward, and can usually be accomplished by three or four people in one day.

Tools and supplies

A backhoe is needed only for installation of the two tanks. All other excavation is done with a small trenching machine that will excavate a cut four to six inches wide. A transit or similar instrument is necessary for staking out the lateral lines on sloping lots. Other tools needed for installation are:

- Shovels, wheelbarrows—for moving gravel
- Electric drill (with power pack or generator, if necessary)—for drilling holes in lateral lines
- Drill bits
- Hack saw, extra blades—for cutting PVC pipe to required lengths
- PVC glue (and rags)
- Mortar—to seal tank openings
- Measuring tape
- Electrical wiring tools

In addition to tools, a complete list of parts and materials should be compiled from a sketch of the system (See Appendices 2 and 3).

Site preparation and imported fill

One of the most important concerns for an LPP system is to protect the site from soil disturbance by heavy equipment. Removal or compaction of the topsoil, especially during wet weather, may destroy the site's suitability for an LPP. As soon as the absorption area has been designated, it should be flagged, roped off and "quarantined" from construction traffic. No site preparation or LPP construction work should occur if the soil is wet. As a rule of thumb, if the soil is too wet to plow, it is too wet to disturb for system construction.

After the location is staked out and the soil is dry enough to plow, the site should be cleared of brush and small trees. If larger trees are removed, they should be cut off rather than uprooted in order to avoid creating depressions and damaging the soil-pore network.

Provisions must be made for intercepting or diverting surface water and shallow groundwater away from the absorption area, septic tank and pumping chamber. This can be done with grassy swales, open ditches or curtain drains.

If the site requires imported fill to improve surface drainage, it must be incorporated evenly into the underlying natural soil. It is very important that no sharp interface remain between the natural and imported soil layers. Before applying the imported fill to the absorption area, the ground surface must be tilled with a small plow or cultivator. Fill should be applied with a minimum of wheeled traffic on the area, and the area tilled again to ensure even mixing. A very small tractor should be used to spread the material around and to provide a convex shape to the area. There should be no low spots or depressions, and the final shape should shed, rather than accumulate rainwater. Use of fill to supplement the soil profile is discussed in Chapter 8.

After the area has been cleared and shaped, the location of the lateral lines and supply manifold should be accurately staked out according to design specifications. Each lateral line must be laid out along a level contour using a transit. One lateral may be higher or lower than the next one, but each individual lateral must be level. In no case should a lateral line be allowed to slope away from the manifold.

Tank installation

The two-compartment septic tank is installed in the same way as a conventional system. Wastewater from the house flows directly into the large compartment of the tank. The pumping chamber is installed next to the septic tank, but its direction must be reversed so that the tee end becomes the inlet end adjacent to the septic tank. The lower invert of the tee end ensures proper gravity flow from the septic-tank outlet into the pumping chamber. The tanks are connected with an appropriate length of solid, four-inch PVC pipe. Inlet and outlet openings around the pipe must then be sealed with mortar.

The tank access lids must be equipped with water-tight masonry or concrete risers to at least six inches above grade. These provide easy access for repair and inspection, and help keep surface water out of the tanks.

If an LPP is being installed to replace an existing conventional septic system, only one additional tank (the pump chamber) must be installed. However, the existing septic tank must be pumped out before installing the LPP.

Supply Manifold

The supply manifold conveys effluent from the pump to the distribution laterals. Any effluent remaining in the lateral lines when the pump shuts off should drain back to the pumping chamber through the supply manifold (unless the system is large enough to require a check valve). The manifold joins each lateral through a short riser pipe connecting a reducing tee on the manifold to a 1¼-inch elbow or tee on the lateral (Figure 6). This assembly places each lateral pipe about six inches higher than the supply manifold and helps prevent the back-flow of effluent from a higher lateral to a lower lateral. The individual riser units may be assembled earlier and glued in place between the laterals after the manifold is cut into segments. Because the lateral line is now several inches higher than the manifold, the manifold requires a trench six inches deeper than the laterals. In the special case of pumping downhill, the laterals are placed lower than the manifold (See Chapter 7).

After the supply manifold has been placed in its trench and lateral lines connected, it should be backfilled with tightly tamped soil. The supply-manifold trench must not be backfilled with gravel, or the trench may become a conduit for downslope flow of effluent from the laterals. The outlet hole in the pumping tank should not be sealed with mortar until after the pump is in place.

Lateral lines

The lateral trenches are usually cut 18 inches deep. Some soil profiles will require shallower placement. The depth of a given lateral trench should be uniform from the manifold to the end of the lateral. In no case should the trench bottom be allowed to slope away from the manifold. The lateral trench must not extend more than one or two feet beyond the end of the lateral pipe. Small earthen dams are placed at the beginning of each lateral trench, and at 20-foot intervals thereafter, to help maintain uniform distribution of effluent along each trench. The dams can be tamped into place or left uncut from the soil (Figure 8). Lateral trench bottoms are then lined with three to six inches of gravel (remember to put no gravel in the supply manifold trench).

The 1¼-inch PVC pipes should be laid out and cut to proper lengths for the lateral lines. Holes are drilled (in a straight line) according to the design specifications after the laterals have been cut to their proper length. The first hole in each lateral should be drilled two to three feet from the manifold; the last hole should be drilled two to three feet from the end of the lateral. Holes must not coincide with the earthen dams. Holes are only drilled through one side of the pipe. If the drill bit should go through both sides, or if a hole is drilled in the wrong place, it can be sealed by wrapping with duct tape. Lateral pipes are placed holes-down in the trenches. A short turn-up with a capped end is at the end of each lateral (Figure 8). The capped end must be brought up above or flush with the final grade. As the trench is backfilled, the turn-up may be placed inside a short length of four- or six-inch PVC or terra cotta pipe to protect it from lawn mower damage, while still providing easy access. When installing each lateral, care must be taken to ensure that the holes are down and the turn-up pointed upward before the quick-drying PVC glue hardens. Positioning of the lateral should be checked to make sure it is level in the trench.

After the lateral lines are in place and leveled, they are covered with another two to four inches of gravel. The earthen dams in the lateral trenches and near the manifold must be tightly tamped from the trench bottom to the ground surface. Finally, the trenches are backfilled with topsoil. Turn-ups should then be cut to appropriate lengths, fitted with caps and (if desired) protected with short segments of four- or six-inch PVC or terra cotta.

Pump and controls

Details of pump installation are show in Figure 7. The pump must be placed on two concrete blocks set next to each other on the bottom of the tank. This prevents the pumping of any solid particles which can clog the LPP system. A piece of nylon rope or other non-corrodible material should be attached to the pump and to the outlet pipe for lifting the pump in and out of the chamber. (The PVC outlet pipe is too fragile to support the pump).

Controls are fastened to the outlet pipe with clamps or brackets supplied by the manufacturer. The lower level control or "pump-off" must be positioned above the pump, so that the pump remains submerged at all times. The upper level control "pump-on" is positioned to pump a specified volume of effluent (Chapter 4). The high-water control float is then mounted about three inches above the upper pump-on control. (**Note:** Care must be taken to ensure that the floats do not become fouled by other components in the tank such as the electric power cord or the lifting rope.)

The pump outlet pipe should be connected to the supply manifold with a threaded PVC union to allow quick removal. The gate or globe valve must also be installed in the supply line (within the pump chamber) to allow final adjustment of the pressure. If effluent will be pumped downhill, a ¼-inch siphon-breaker hole must be drilled in the bottom of the supply line before it leaves the pump tank. This breaks any vacuum in the system and prevents the inadvertent siphoning of effluent out of the tank. This hole is very important.

Power and control cords should be guided out of the pump chamber through a recessed channel or opening that will protect the cords from damage by the concrete lid.

Electrical connections

As noted earlier, the pump and high-water alarm must be placed on separate electrical circuits. (If the pump circuit fails, the alarm must still be able to operate). Follow the manufacturer's recommendations for proper fuses or circuit-breakers.

All electrical connections must be made outside the pumping chamber. Power cords from the pump and controls should be plugged into a NEMA-approved outdoor receptacle mounted outside of the pumping chamber. The receptacle must not be located inside the pumping chamber due to the corrosive and explosive gases that may form from the sewage.

Electrical connections may be made inside the pumping tank only if wired inside a sealed, watertight box. Some level-control switches have such a

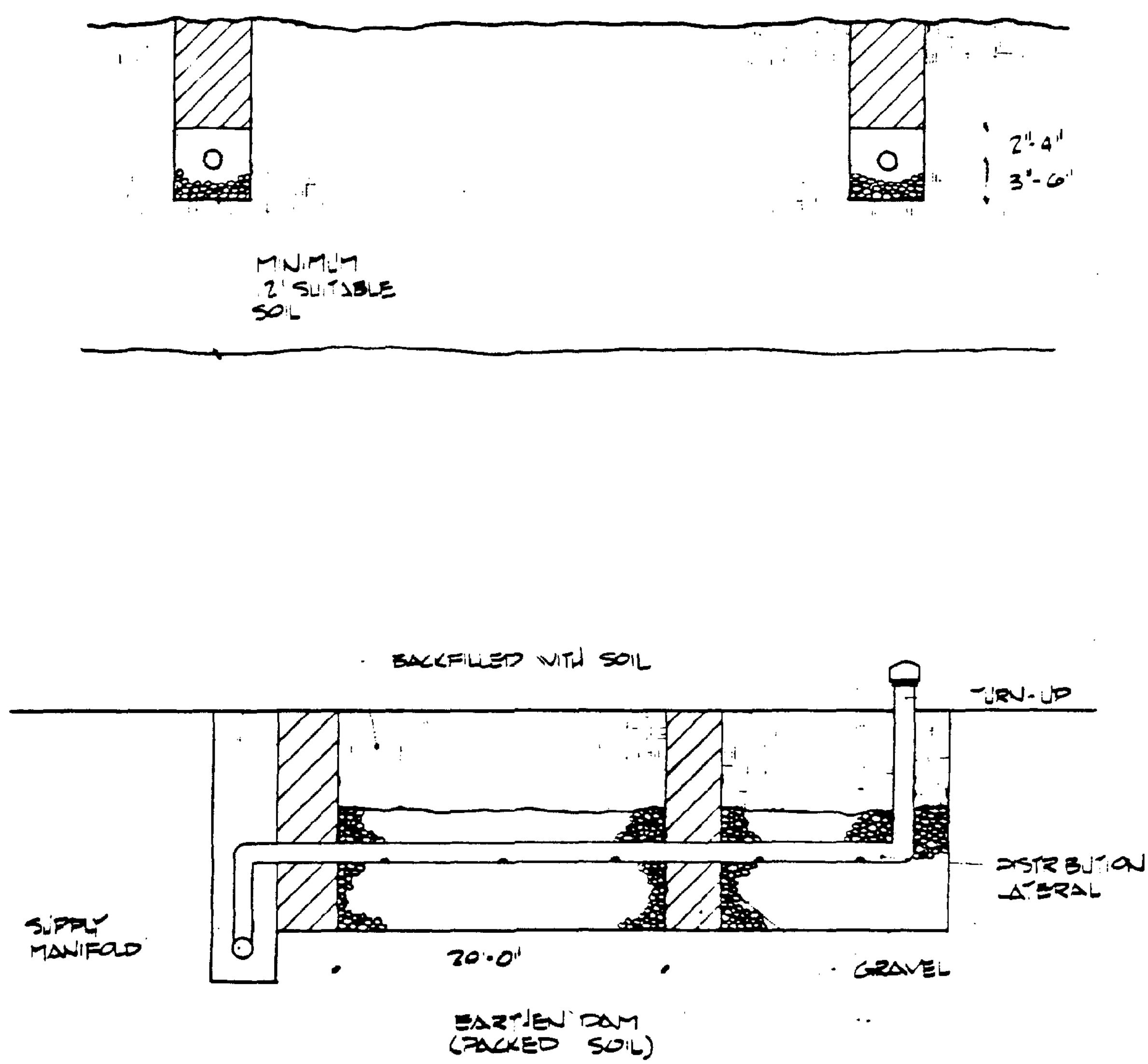


Figure 8. Details of absorption trenches

box built into the housing but are more expensive than the plug-in devices.

Wiring between the pumping chamber and the house should meet state and local code requirements. A lightning arrestor is recommended to protect the pump and controls from electrical surges.

Proper operation check

After all components have been installed and connected, the system should be checked for proper operation. With electrical power turned off, fill the pumping chamber with a garden hose (or allow effluent to accumulate) until the liquid rises to the level of the high-water alarm float.

Turn on the electrical power. The alarm light should go on in the house, and the pump should start operating. The alarm light should go off when the liquid level falls below the high-water float. The pump should turn off when the liquid reaches the lowest float control. Be sure the pump is still completely submerged.

Pressure head adjustment

The pressure head must be adjusted to match that specified in the design. The pressure head is measured as the height liquid will rise above the turn-up elbow when the pump is running. To adjust the head:

- Glue a four-foot length of pipe (preferably clear) to a threaded adapter that will screw onto the turn-up adapters.
- Replace the turn-up cap with the pipe and adapter.
- Turn the power on to allow liquid to rise in the pipe.
- Adjust the gate or globe valve in the pumping tank until the effluent reaches the desired height in the pipe. Remember to include the distance below the ground surface to the lateral line when measuring the height.

Final landscaping

After the LPP is installed, the following should be checked to ensure that the system will not be overloaded with excess rainwater and runoff:

- The distribution field is shaped to shed rainwater and is free of low areas.
- Curtain drains, grassy swales or ditches for diverting ground and surface water are properly installed.
- Gutter and downspout drains are directed away from the system.

Any problems should be corrected before approving the system.

Finally, the entire area should be planted with grass in order to prevent erosion. The soil should be properly tilled, limed (if necessary) and fertilized before planting. After applying an appropriate grass seed, the area should be heavily mulched with straw or other suitable material.

CHAPTER 7

LPP Design and Installation on Sloping Ground

A sloping site presents a special set of problems for LPP design. The system must be carefully planned to obtain even distribution of effluent throughout the absorption area. The pressure head on each line is different due to a different elevation. Each foot of elevation difference changes the pressure head by one foot. Also, perched water moving downslope onto the system and effluent moving from the upper trenches to the lower trenches can cause overloading. Pumping uphill or downhill to the absorption field can create additional problems. This chapter highlights changes in the design procedure which are necessary when designing LPP systems on slopes.

Layout

The procedure is similar to that in Chapter 3, with careful emphasis placed on the following points:

- Lateral trenches must be placed on contour and earthen dams installed as needed to ensure even distribution of effluent in each trench (Figure 9).
- The effects of slope can be lessened by making systems as long and narrow as possible across the contour (Figure 2C, page 6). This design uses fewer and longer lines, decreasing the elevation difference between the highest and lowest lines.
- Systems with more than four feet of elevation difference between the highest and lowest laterals cannot be designed with a single manifold. Separate manifolds for the upper and lower lines must be used (Figure 9b). Each manifold must have its own pressure-control valve (gate or globe) for pressure adjustment.
- Interceptor or curtain drains are often necessary to divert water moving from uphill.
- When it is necessary to pump downhill, distribution lines should be in deeper trenches than the supply manifold. The opposite is true for level or uphill systems (Figure 10).
- Installation on slopes greater than 30 percent is not recommended unless installation is to be done entirely by hand.

Dosing and distribution

The design must compensate for differences in elevation head in order to achieve uniform distribution. The load on each line must be individually calculated. All the loads are then balanced by modifying the design of individual lines where needed.

Determine dosing rate:

Step 1. Measure and record the elevation of each line. Make sure that each line is laid out on the contour (see example below for summary of steps).

Step 2. Round-off each elevation to the nearest half-foot.

Step 3. Compute the difference in elevation of each line from the highest line.

Step 4. Determine the pressure head on each line. First select the pressure head for the highest line. Then add the elevation difference (Step 3) to determine the pressure head on the lower lines.

Example:

Calculate the pressure head on each line for a system with five 60-ft lines with elevations shown below. Pressure head for the highest line is 2 ft. See Table 5 below.

Table 5. Calculating pressure head

Line	Elevation (Step 1)	Round Off (Step 2)	Difference (Step 3)	Pressure Head (Step 4)
	ft			
1 Highest	359.2	359	0	2
2	358.6	358.5	0.5	2.5
3	358.2	358	1	3
4	357.9	358	1	3
5 Lowest	357.0	357	2	4

The pressure head should not exceed five feet on any of the lines. If it does, several modifications can be made. If suitable space is available, redesign the system, making it longer and narrower, thus covering less of a range in elevation. Remember that the lateral length is restricted to 70 feet or less, and the spacing to five feet or more.

As another option, lower the selected pressure head on the highest line and recalculate the heads on the remaining lines. The head on the highest line should be no less than one foot and is best kept at two feet.

Finally you can split the line into two or more manifolds. This is discussed in detail later in this chapter.

Step 5. Check to see if the pressure head exceeds five feet on any lines.

Example:

Highest pressure head is 4 ft, therefore no modifications need to be made.

Step 6. Determine the flow rate per hole for each line using Table 2 (pg 8) and the pressure heads calculated above. (See following example.)

Step 7. Determine the flow rate for each line.

Example:

Using the pressure heads above and assuming a 5-ft hole spacing on 60-ft lines (12 holes/line), prepare Table 6 below.

Table 6. Flow rate for each line

Line	Pressure Head (Step 4)	Flow Rate/Hole (Step 6)	Flow Rate/Line (Step 7)
	ft	*gpm*	*gpm*
1	2	.41	4.9
2	2.5	.46	5.5
3,4	3	.50	6.0
5	4	.58	7.0

The dose to the lower lines is larger due to the increased pressure head, while the dose to the upper lines is reduced, causing overloading of the lower lines. The flow rate should be balanced to within 10 percent among lines on the same manifold. It is wise to reduce the flow even lower in the lowest lines, because they receive an additional hydraulic load from downslope effluent movement from the upper lines.

Often the lengths of lateral lines vary. Some may be shorter than others to avoid obstacles such as large trees, rocks or complex slopes. When this is the case, the flow rates of the lines cannot be directly compared. Rather the flow rates per foot of line must be calculated and these compared.

Step 8. Balance flow rate among lines. This can be done either by changing the number of holes or changing the size of the holes. The flow to lower lines can be reduced by increasing the hole spacing to greater than five feet or reducing the hole size to as small as 3/32 inch. But these sizes and spacings must not be used for an entire system.

Example:

For the system in discussion, change the hole spacing to 4 ft in line 1 (highest) and to 6 ft in line 5 (lowest).
See Table 7 below.

Table 7. Balancing the flow rate among lines

Line	Hole Spacing	No. of Holes	Flow/Hole	Flow/Line*
	ft		*gpm*	*gpm*
1	4	15	.41	6.2
2	4.5	13	.46	6.0
3,4	5	12	.50	6.0
5	6	10	.58	5.8

*For systems with lines of variable length, the flow rate/ft is compared as described in Step 7.

When changing hole size or spacing to balance the flow it is very important to make the changes and instructions simple and clear. Hole placement and line installation should be inspected to ensure that they are done properly.

Step 9. Calculate total dosing rate. The dosing rates for each line are added to obtain the total.

Example:

For the system above:
Dosing rate = 5.8 + 6.0 + 6.0 + 6.0 + 6.2 gpm
= 30.0 gpm
(Add 2 gpm if a siphon-breaker hole is needed.)

Pump selection

The pump is chosen in the same manner as in Chapter 4. When pumping uphill the elevation head increases. If the hill is large enough it may become impractical to adjust the system for use with a 4/10-horsepower pump. It may be necessary to use a larger, more expensive pump.

If it is necessary to pump downhill, a ¼-inch siphon-breaker hole must be drilled in the supply line in the pumping tank (Figure 7) to avoid unintentional continuous siphoning of effluent from the tank to the absorption field.

In some downhill systems, intentional siphoning can be used instead of pumping to provide distribution. A gravity-dosing siphon replaces the electric pump. Siphons of different sizes are available, and the siphon and dosing volume must be matched. The remainder of the system

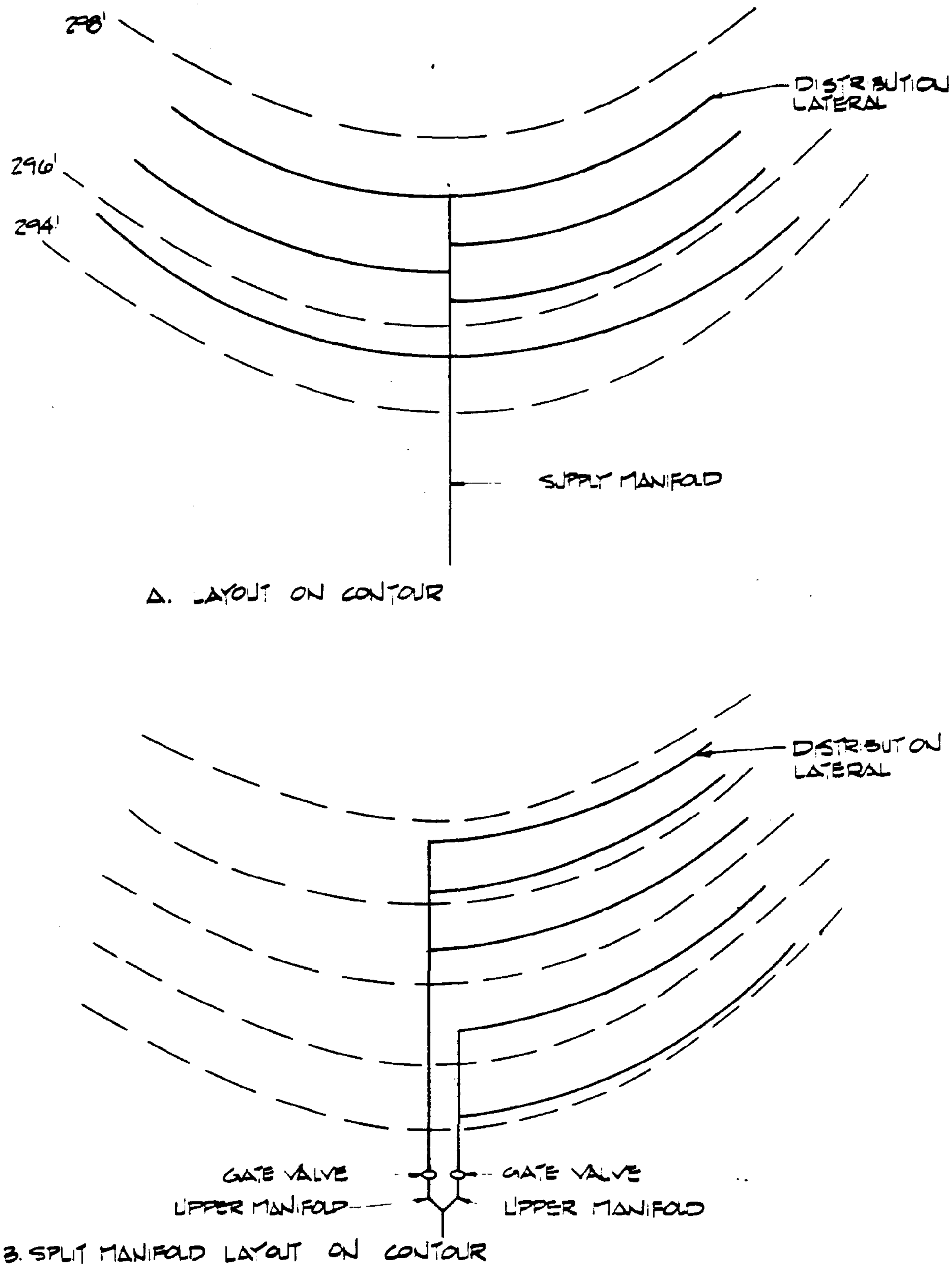


Figure 9. Layout of LPP systems on slopes

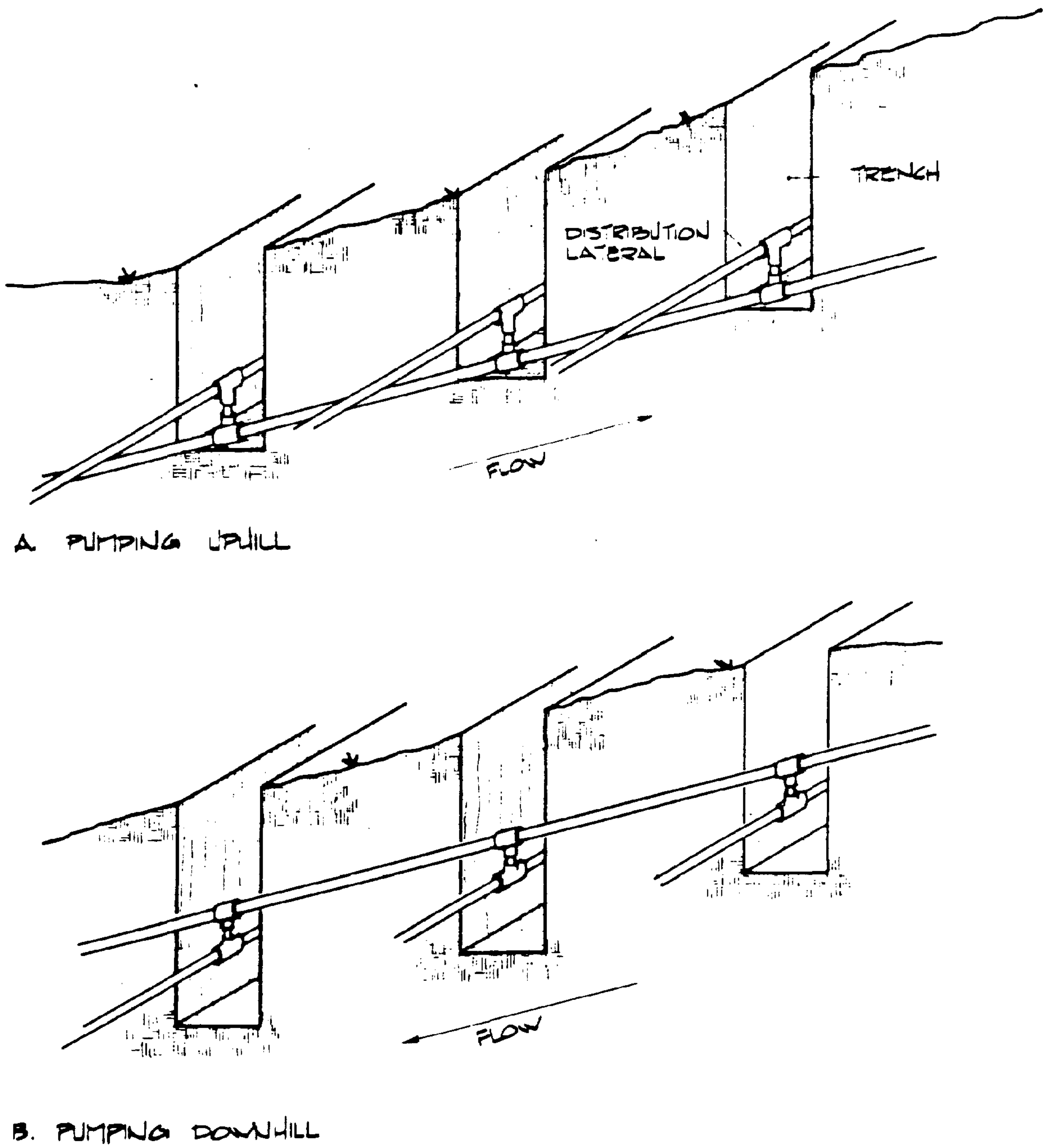


Figure 10. Manifold placement on slopes

design is the same as when a pump is used.

The remaining steps in the design of LPP systems for sloping ground are the same as that for level ground (Chapter 4).

Design of split manifold systems

A split manifold system is used when the elevation difference between the lowest and highest lines exceeds four feet. The supply line is split into two or more manifolds, each connected to a subsystem of distribution laterals (Figure 9b). Each manifold is equipped with a gate or globe valve so the pressure heads on the subsystems can be adjusted separately. This allows each subsystem to act as an independent system although they may be operated from the same pump. The following is an example of a design where a split manifold is necessary.

Example:

Lines are to be laid out on contours at 1319.8, 1318.4, 1317.0, 1315.2, 1313.7 and 1312.4 ft.

Steps 1-5. The procedure in the previous section is followed. The calculations are summarized in the following example.

Example:

Pressure head of highest line is set at 2 ft. See Table 8 below. The pressure head exceeds 5 ft for 3 lines; therefore a split manifold will be used.

Table 8. Calculating the pressure head

Line	Elevation (Step 1)	Round Off (Step 2)	Difference (Step 3)	Pressure Head (Step 4)
	ft			
1	1319.8	1320	0	2
2	1318.4	1318.5	1.5	3.5
3	1317.0	1317	3	5
4	1315.2	1315	5	7
5	1313.7	1313.5	6.5	8.5
6	1312.4	1312.5	7.5	9.5

Step 6. Split the system into two subsystems.

Example:

Subsystem 1 (higher) = lines 1-3
Subsystem 2 (lower) = lines 4-6

Step 7. Repeat steps 1 through 5 independently for each subsystem.

Example:

Set the pressure head at 2 ft for the highest line of each subsystem. See Table 9 below. No pressure head exceeds 5 ft; therefore this system is satisfactory.

Follow the procedure of steps six through nine in the previous section to balance the flow rates and determine dosing rates.

Pump selection is done as in Chapter 4. When using a split manifold, the total friction loss decreases while the pipe volume increases. In many cases it may be best to decrease the diameter of the manifolds after they split. This will decrease the pipe volume, and may avoid the need for a check valve.

For most systems the gate or globe valves should be 1¼-inch diameter because they are easier to adjust than larger valves. Reducing adapters will be needed to fit these valves into larger diameter manifolds.

Table 9. Establishing a subsystem

Subsystem	Line	Elevation and Round Off (Steps 1,2)	Difference (Step 3)	Pressure Head (Step 4)
		ft		
1	1	1320	0	2
	2	1318.5	1.5	3.5
	3	1317	3	5
2	4	1315	0	2
	5	1313.5	1.5	3.5
	6	1312.5	2.5	4.5

CHAPTER 8

Modified LPP Systems Using Fill

Most sites with a restrictive horizon or a seasonally high, water table within 24 inches of the surface are not suitable for a standard LPP system. Many are not suitable for any soil-absorption, waste-treatment system. But some of these sites can be used for waste treatment if the soil is supplemented with fill that has been carefully selected and added.

When there is 16 inches to 24 inches of usable soil on an acceptable site, a modified LPP can be installed. The soil must have suitable or provisionally suitable texture, structure and permeability (10 NCAC 10A .1920). After the addition of fill, trenches are placed as shallowly as three inches to four inches into the natural soil. The design and installation of the modified LPP are discussed below.

When there is less than 16 inches of usable soil, a mound system can be built, where the distribution lines are placed above the soil surface in imported fill. The design and construction of mounds is discussed in the manual, *Design and Installation of Mound Waste Treatment Systems* published by UNC Sea Grant.

Modified LPP design

The only difference between designing a modified and standard LPP is the calculation of the fill requirements. The volume of the fill needed is the area to be filled multiplied by the depth of fill. The area to be filled is the absorption field plus a five-foot buffer around the edges.

Step 1. Calculate area to be filled. Add 10 feet to the length and width of absorption area to allow for buffer space.

Example:

For a 60 ft x 30 ft absorption field to be filled 1 ft deep:
Total area = 70 ft x 40 ft

Step 2. Calculate the volume of fill needed.

Example:

V fill = total area x depth of fill
V fill = 70 ft x 40 ft x 1 ft
= 2800 ft^3

Step 3. Convert to cubic yards.

Example:

V fill = 2800 ft^3/27 ft^3 per yd^3
= 104 yd^3

The remaining design steps follow the procedure of Chapters 3 and 4.

Installation

The success of a modified LPP depends on the care used in selecting and incorporating the fill material. The fill must have a sandy loam or loamy sand texture. The fill should not be hauled or worked wet.

As with all LPP systems, the site must be protected from traffic. Prior to incorporating the fill, brush and small trees should be removed and the soil suface loosened using a cultivator or garden plow. It is very important that the soil be worked only when dry. Working damp or wet soil can cause compaction and sealing, leading to failure of the system.

Fill is moved to the system using a front-end loader, being careful to avoid driving on the plowed area. The first load of fill is pushed into place using a very small crawler tractor with a blade or a roto-tiller with a blade. The fill is then tilled into the first few inches of natural soil to create a gradual boundary between the two. Failure to do this could ruin the system by forming a barrier to water movement at the soil-fill interface. Subsequent loads of fill are placed on the system and tilled, until the desired height is reached. The site should be shaped to shed water and be free of low spots before proceeding.

To install the LPP follow the procedure discussed in Chapter 6.

CHAPTER 9

Inspection and Maintenance

The successful performance of an LPP relies on proper design and installation. The details for a given system, from site preparation to final landscaping, should be carefully specified on the Improvements Permit. This helps clarify the responsibilities of the property owner, contractor and permitting agency and helps avoid last-minute surprises when issuing a Certificate of Completion. Items on the Improvements Permit (and associated design specifications for the LPP system) should be inspected by the permitting agency in four stages as outlined in Appendix 4.

Installation inspection

Regulatory agencies are strongly recommended to withhold the Certificate of Completion until all the above requirements are satisfied. A checklist similar to Appendix 4 should be completed and filed each time a system is installed to ensure completion of the requirements.

Operation inspections

A properly designed and installed LPP system requires very little maintenance. Several routine items should be checked periodically and an extra pump should be readily available. LPP systems should be observed by the regulatory agency one, three, six and nine months after initial installation, and every six months thereafter. An inspection report should be completed and filed each time the system is checked. A sample format is shown in Appendix 5.

Routine maintenance

All septic tanks, whether for conventional or alternative systems, require occasional pumping. Sludge and scum accumulation should be checked annually. Virtually all solids will be retained in the first compartment of the two-compartment septic tank. Little or no accumulation should occur in either the second compartment of the septic tank or in the pumping chamber. The rate of sludge accumulation will vary with individual living habits. Most septic tanks require pumping about once every four years.

Some LPP systems may gradually accumulate solids at the ends of the lateral lines. These should be removed at least once a year by unscrewing the caps on each of the turn-ups, and back-flushing the laterals with a garden hose.

Pressure head in the upper and lower laterals should also be checked and adjusted one month after installation and annually thereafter (Chapter 6). Proper pump and float-control operation should be checked during all routine inspections. If the alarm panel has a "push-to-test" button, it should be checked regularly. Pump maintenance should follow the manufacturer's recommendations.

Repair procedures

The alarm light should go on whenever effluent in the pump chamber rises above the pump-on level control. This can occur for several reasons:

- Power failure: If there has been a power failure, effluent will continue to accumulate in the tank until power is restored. At this time the alarm may come on for a brief period (less than 30 minutes), but will go off as soon as the pump draws down the effluent.
- Pump or switch failure: If the pump or level controls malfunction, they can be quickly replaced by unscrewing the PVC union and lifting the entire assembly out of the pumping chamber (use the nylon lift rope). Be sure to turn off the power supply, and disconnect all cords before removing or replacing the pump or control assembly.
- Clogged valve or discharge holes: If the distribution system becomes clogged, the tank will not be emptied. Back-flush the laterals and supply manifold if necessary.

Before replacing any components, make sure that the level controls have not simply become tangled. The problem can usually be isolated by checking the pump operation independently from the controls. Repair or replace the appropriate components.

Appendix 1. Design specifications for example LPP (Chapters 3 and 4)

File a copy of this sheet along with an accurate sketch for each LPP designed.

Daily waste flow	450 gal
Septic tank size	1200 gal
Pumping tank size	900 gal
Effluent loading rate	0.25 gal/ft^2/day
Absorption area	1800 ft^2
Total length of laterals	360 ft
Lateral diameter	1¼ in.
Lateral configuration	6 x 60 ft lines
Supply line length	70 ft
Supply line diameter	2 in.
Manifold placement	side
Hole size*	5/32 in.
Hole spacing	5 ft
Number of holes	72
Pressure head	3 ft
Flow per hole	0.50 gpm
Total flow	36 gpm
Elevation head	5 ft
Friction head	1.8 ft
Pressure head	3 ft
Total head	9.8 ft
Pump requirements	36 gpm, 9.8 ft of head
Storage volume in laterals	32.4 gal
Storage volume in supply line	13.6 gal
Total storage volume	46.0 gal
Dosing volume	180 gal
Dosing depth	10 in.
Check valve needed?	No

*Data on hole size, spacing, pressure head and flow must be listed for each line for systems where lines are different (such as sloping lots).

Appendix 2. Pipe and fittings for example LPP (Chapters 3 and 4)

Type	Size	Quantity	Description
Pipe, 160 psi	4 in.	10 ft	Connects septic tank to pumping tank
Pipe, 160 psi	2 in.	70 ft	Supply manifold
Pipe, 160 psi	1½ in.	10 ft	Connects pump to supply manifold
Pipe, 160 psi	1¼ in.	380 ft	Laterals plus extra length for turn-ups
Tee*	2 x 2 x 1¼ in.	5	For joining manifold to first 5 laterals
Elbow	2 x 1¼ in.	1	For joining manifold to last lateral
Elbow	1¼ in.	12	6 for joining laterals to manifold 6 for turn-ups
Male adapter	1¼ in.	6	For turn-ups
Threaded cap	1¼ in.	6	For turn-ups
Male adapter**	1½ in.	3	1 for pump outlet 2 for gate valve
Elbow	1½ in.	1	For pump to supply line connection
Bushing	1½ x 2 in.	1	For pump to supply line connection
Threaded union	1½ in.	1	For quick removal of pump
Gate valve	1½ in.	1	PVC or brass
PVC glue	1 qt	1	
PVC primer	1 qt	1	

*Details of these connections are shown in Figures 5 and 6.

**Size of this adapter and the following fittings depend on size of pump outlet.

Appendix 3. Other supplies for example LPP

Type	Size	Quantity	Description
Pump	0.4 hp	1	Submersible effluent pump
Switch		1	Sealed level controls adjustable to 10 in. drawdown
Alarm		1	Sealed mercury float switch and alarm light
Wiring			Approved outdoor receptacle, wire and conduit for 110V service
Septic Tank	1200 gal	1	Two compartment
Pumping Tank	900 gal	1	Single-compartment septic tank
Risers		2	Concrete risers or well tiles, or blocks and mortar—to raise tank lids six in. above final grade
Lids		2	To fit on risers
Gravel	⅜-1 in.	5 yds.	Washed
Concrete blocks		2	Raised support for pump
Nylon rope		8 ft	To remove pump from tank
Mortar			To seal around supply line and riser
Grass seed			If needed to establish grass cover
Lime			
Fertilizer			
Mulch			

Appendix 4. LPP Construction Inspection Checklist

Site Identification

Site Preparation Date ______

1. Is the site in the right location? ____
2. Roped off and protected from traffic? ____
3. Small trees and brush cleared? ____
4. Provisions for site drainage? ____
5. Fill incorporated with underlying soil? ____
6. Distribution field shaped to shed water? ____
7. Lines staked out properly? ____
8. Comments ______

______

Construction Check Date ______

1. Tanks:
   - Proper size and type? ____
   - Installed properly? ____
2. Manifold and laterals:
   - Depth of gravel suitable? ____
   - Placement of dams? ____
   - Holes drilled properly and placed downward? ____
   - Manifold and laterals connected properly? ____
3. Water conservation devices installed in house? ____
4. Comments ______

______

Operation Check Date ______

1. Pump and switches operating? ____
2. High water alarm operating? ____
3. Electric receptacle outside pump tank? ____
4. Pressure head in lateral lines?
   a. Lowest ____
   b. Highest ____
5. Comments ______

______

Final Landscaping Date ______

1. Site shaped to shed rainwater? ____
2. Any low areas? ____
3. Diversion drains? ____
4. Downspout drains directed away from system? ____
5. Seeded and mulched? ____
6. Comments ______

Appendix 5. Maintenance Checklist

Site Identification Date ______

System Type ______

Site Examination

1. Any rainfall in last 3 days? ______
2. Effluent ponded on surface? ______
3. Indications of recent ponding? ______
4. Ground above system damp and mushy compared to surrounding area? ______
5. Noticeable odor of sewage? ______
6. Other

If any "Yes" answers, sketch location and extent on back of page.

Site Maintenance

1. Condition of vegetable cover ______
2. Site drainage (roof water, ditches, etc.) ______
3. Riser and lid ______
4. Turn-ups ______
5. Erosion ______

Pump Examination

1. Pump and switch properly plugged in? ______
2. Pump operating? ______
3. Switch operating? ______
4. Good seal where supply line leaves tank? ______
5. Quality of effluent
   - Greasy? ______
   - Sludge accumulation? ______
6. Measure pressure head and adjust.
   - Initial head ______
   - Adjusted head ______
7. Comments on problems noted above. ______

______

______

Comments From Homeowner. ______

______

Additional Observations. ______

______

______

______

______

______

______

WASTE WATER SYSTEMS, INC.

"PERC-RITE"™

WASTE WATER DISPOSAL SYSTEMS

Design Guidelines and Manual

(Residential Sized Systems)

Rev. 9-92

TABLE OF CONTENTS

TABLE OF CONTENTS
(Cont.)

PART ONE

Description of the "Perc-Rite"™ Waste Water Disposal System

The "Perc-Rite"™ Waste Water Disposal System is an improved soil absorption system for waste water disposal. A soil absorption system must serve two purposes: 1) keep untreated effluent below the surface, and 2) treat the effluent before it reaches ground or surface water. The system works best when the distribution area is not saturated with water or effluent, allowing efficient aerobic bacteria to treat the wastes.

There are several conditions which frequently hinder the operation of soil absorption systems. Clogging of the soil can occur from localized overloading during use or from the mechanical sealing of the soil-trench interface during construction. This clogging can cause effluent to break through to the surface, especially in fine-textured soils. Anaerobic conditions caused by continuous saturation due to overloading or a high water table retard treatment, increasing the potential for pollution. Shallow soils are not deep enough to purify the effluent.

The "Perc-Rite"™ System has five basic design improvements over conventional or L.P.P. type systems to overcome the above problems. The improvements are as follows:

- Uniform distribution of effluent
- Accurate control of effluent emission rates
- Dosing and resting cycles
- Shallow installation of drip lines
- Filtering of effluent before dispersal

Problems from local overloading are decreased when effluent is distributed over the entire absorption area. Dosing and resting cycles help maintain aerobic conditions in the soil, improving treatment. Shallow placement increases the vertical separation from the system to any restrictive horizon or seasonally high water table.

A "Perc-Rite"™ System is a shallow, slow rate pressure dosed soil absorption system. The basic components of which are as follows:

- Septic tank, sand filter or aerobic treatment unit
- Pumping chamber/dosing tank
- "Perc-Rite"™ pump and controls
- "Perc-Rite"™ automatic back flushing filters
- "Perc-Rite"™ pressure compensating waste water drip line laterals
- Supply and return manifold lines
- Alarms
- Suitable soil absorption area

The "Perc-Rite"™ Filtering and Sub-surface Distribution System is operated via a "state of the art" controller which is activated by a sensing device located in a dosing tank downstream from pretreatment. When activated by the level of effluent in the dosing tank, the controller will start the disposal cycle and pump the effluent through a 115 micron disc filter. The filter configuration is modular and can be amplified according to individual system needs. The degree of filtration (microns) may be increased or decreased according to needs.

The inlet manifold carries the unfiltered effluent to the filter. The effluent passes through the filter main valve, then through the filter itself, while the filter back flushing valve remains closed. The filtered effluent flows through the outlet manifold and is discharged below the soil surface through a patented chemical- resisting, pressure compensating "drip" poly-tubing. The construction of the "RAM drip" tubing is unique in that the internal diaphragm and labyrinth provides for an exact amount of effluent to be discharged from each of its emitters which are normally spaced at two foot intervals along its entire length. The "RAM drip" tubing maintains a constant dripper flow over pressure ranges of 5 up to 60 psi. Because the effluent is distributed at a relatively low rate, large quantities of effluent may be distributed over long periods of time without saturating the surrounding soil, thus eliminating the possibility of run-off or surface water ponding. When the back flushing schedule is triggered, the filter valve closes, thus blocking the flow of unfiltered effluent to the filter. After a short delay, the flushing valve opens, thereby discharging the filtered impurities into the collector manifold. The closing and opening procedure of the filter and back flush valves causes a change of flow within the filter. When the filter valve closes, the upstream pressure is blocked, while the downstream pressure reverses the flow of the filtered effluent back through the outlet of the filter. The inverted flow carries accumulated particles from the filter rings through the open back flushing valve through the collector manifold to the influent side of the pretreatment tank. The back flush procedure lasts approximately fifteen seconds, upon where the back flushing valve closes, then the filter valve opens and the inverted flow is cut off. Only after the first filter has completed its flushing cycle, will the second filter begin its cycle of back flushing in the same manner as the first filter. The dripper lines are automatically flushed every 200 dosing cycles. This function is activated by the controller which opens the field flush valve, thus allowing the flushed effluent to be returned to the pretreatment tank. The duration of this cycle is approximately three minutes. The flushing action creates a high velocity of the effluent which produces a cleaning action over the inside walls of the dripper tubing, P.V.C. manifolds and emitters.

The Telemetry Equipment monitors the following functions:

High water alarm (audio)
Power out alarm (audio)
Pump start control
Timed back flush of filters
Automatic flushing of drip system
Flow variance
Remote monitor of alarm functions

In the event of a power outage or a high water condition, an audio alarm will sound. Simultaneously, the service center can be notified via telephone link up through a digital communications receiver. Should a flow variance (plus or minus 20%) occur, a signal will be transmitted to the service center indicating this trouble. A minimum of 24 hours storage capacity is designed into the system should a power failure occur.

Since the field distribution lines require very little soil disturbance and effluent discharge volume from each emitter hole is insignificant, installation of the system has very little site impact even in established lawns and gardens. There are no visible indications that the installation site is being used for disposal purposes. This distribution system will permit waste water disposal in land areas that are used for such purposes as parks, athletic fields, groves and highway rights of way. This system is especially suited for landscaped or wooded areas around buildings, trailer parks, apartment complexes or residential subdivisions.

For existing or new treatment facilities - residential, commercial, industrial or municipal - our filtering and sub-surface distribution system can be a viable alternative to land application techniques (spray) or direct stream discharge.

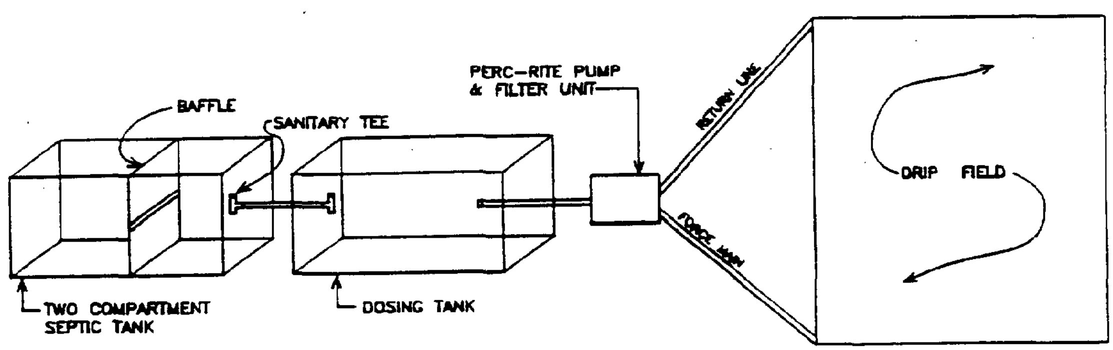


FIGURE 1 - BASIC COMPONENTS OF PERC-RITE SYSTEM

PART TWO

Site and Soil Requirements and Considerations for "Perc-Rite"™ Systems

(General Guidelines)

The suitability of a "Perc-Rite"™ System for a given site is usually determined by soil and its limitations, site slope and landscape characteristics and the available space for absorption systems as well as the anticipated waste water flow. The criteria below is a set of guidelines to follow to practically determine site suitability.

General Space Requirements

The drip field network of lateral lines for most residential "Perc-Rite"™ Systems can occupy anywhere from 1,000 square feet up to 10,000 square feet of area depending on soil texture and permeability and design waste water load. In some cases, according to local regulations an area of equal size may need to be set aside for repair areas. The septic tank, aerobic treatment unit, dosing tank and distribution field are also subject to set back regulations to keep required distances from wells, property lines building foundations and bodies of water according to local regulations. Calculating space requirements will be discussed in Part Three of this manual.

Soil Requirements

A "Perc-Rite"™ System should be situated on the best soil and site on the lot. A minimum of 24 inches of usable soil is recommended between the bottom of the drip line tubing and any underlying restrictive horizons such as consolidated bedrock or hardpan, or the seasonably high water table. In some cases, as determined by the soils engineer or soils scientist for the site, the minimum usable soil depth may be reduced to as little as 12 inches. The "Perc-Rite"™ drip lines may be installed as shallow as 6 inches or less (depending on soil characteristics and freezing depths) making minimum soil depth requirements easier to obtain than when using conventional systems. The usable soil must be of suitable or provisionally suitable texture, structure and permeability as defined in the "Perc-Rite"™ loading rate guidelines or state regulations. In some cases, where the depth to the seasonable high water table or the restrictive horizons is less than recommended, a modified "Perc-Rite"™ System using imported fill may be installed. Great care must be used in building these fill systems. Their design and construction are covered in this manual.

Topography

"Perc-Rite"™ System designed on slopes usually do not require special design and installation procedures. Head losses due to elevation changes should be considered to ensure standard pump sizing will deliver the required flow and pressure. Since the "Perc-Rite"™ pressure compensating dripper emission rates are consistent at varying pressure differences, no special design requirements to ensure proper soil loading rates are needed. Normal spacing between dripper line laterals is 24 inches. On severe slopes of 20% or more, the dripper line laterals may need to be spaced wider than normal due to gravitational effects on water movement. Topography considerations are further discussed in this design manual.

Drainage Requirements

Depressions, gullies, drains and erosional areas must be avoided to prevent hydraulic overloading by surface runoff. Neither the septic tank, pumping chamber nor distribution field should be located in such areas. Surface water and perched groundwater must be intercepted or diverted away from all components of the "Perc-Rite"™ System. Site modifications to the "Perc-Rite"™ System area may be required to ensure all surface or perched water will be intercepted.

PART THREE

Layout of the "Perc-Rite"™ System

The next few parts of this manual are a step by step procedure for correctly designing a "Perc-Rite"™ System. There is no single "Perc-Rite"™ System layout or unit that fits all sites - each must be designed individually.

Size of the Absorption Area

The total amount of absorption area required generally depends on two factors: 1) the daily waste water flow of the facility or home being serviced by the system and 2) the absorption capacity of the soil.

STEP 1:

Calculate the estimated daily flow. For residential systems, usually the local health department will assign a flow in gallons for every bedroom in the house.

Example: Assign flow of 150 gallons per day per bedroom for a three bedroom house.

Flow = 150 GPD x 3 Bedrooms = 450 Gallons Estimated Daily Flow

STEP 2:

Determine the loading rate.

A field evaluation of the soils at the site must be completed by a qualified person (i.e. soil scientist). The data gathered at the site should include all the information necessary to completely fill out the Site Evaluation Sheet as shown (see Soil Form on Page 15 and 16). Upon review of the completed Soil Information Sheet and a site investigation, a maximum hydraulic loading rate is established using Table 1 (the USDA Soil Classes and Maximum Loading Rate Table on Page 17). Table 1 shows the <u>Maximum</u> Hydraulic Loading Rate that should be used for each soil group. See the notations at the bottom of Table 1. Chart 1 (See Page 18) is an example of how soil textures are determined on the bases of clay percentage. Maximum loading rates will be stated in gallons per day per square feet. Note: In fill systems, soil fill must be closely monitored and evaluated as discussed later in this manual.

Example: Determine a loading rate. Soil investigation shows Class III - Sandy Clay Loam (SCL) use a maximum .15 gallons per square feet per day loading rate. Let's assume in this particular case that land room is not restrictive so we can use an even more conservative loading rate as a safety factor - (SCL) carries a maximum .15 loading rate. So we will use a .1 gallons per square feet per day loading rate.

STEP 3:

Compute total area necessary for the absorption field. Use the following equation:

$$\text{Area necessary} = \frac{\text{Daily Flow}}{\text{Daily Loading Rate}}$$

Example: Using the flow as calculated in Step 1 and Step 2.

$$\text{Area necessary} = \frac{\text{450 GPD}}{\text{.1 Gal/Day/Sq.Ft.}} = \text{4,500 Square Feet}$$

STEP 4:

Determine the total length of dripper line required in the absorption area. Spacing between dripper lines is normally 24 inches. The equation to determine total length of dripper line required would then be:

$$\frac{\text{Total Absorption Area in Sq. Ft.}}{\text{Dripper Line Spacing}} = \text{Length of Dripper Line}$$

Example:

$$\frac{\text{4,500 Square Feet}}{\text{2 Foot Spacing}} = \text{2,250 Linear Feet of Dripper Line Needed}$$

STEP 5:

Determine layout and shape of the dripper line absorption field.

When selecting the best layout and shape of the dripper line absorption field you must always place the dripper lines along the contours of the ground area, keeping the dripper line lateral runs as close to the same grade as possible. However, a completely level lateral run is not required. Also, always try to keep each individual dripper line length no greater than 400 feet from its connection to the supply manifold to its connection to the return flush manifold because of excessive friction loss. Some examples of dripper line layouts are shown in Figure 2 (see Page 9) and Figure 3 (see Page 10). Always configure the system supply line to feed at the lower end of the dripper field lines and return from the highest elevations.

When running a continuous dripper line that may turn and make a loop or series of loops back to the return flush line before making a connection, make a transition to solid tubing that will resist kinking and does not emit effluent in the turn, as shown in the example in Figure 4 (see Page 11) and Figure 5 (see Page 12).

IMPORTANT:

In many cases where dripper line lengths exceed a total of 1,000 feet or more it will become necessary to split the absorption field into at least two zones. This is required because a large system with over 1,000 feet of dripper line or more has an operating flow rate that will probably exceed the capabilities of the standard "Perc-Rite"™ unit. By using two or more zones, this problem is easily overcome and is described more in depth later in this manual. (See Drip System Dosing and Design.)

REMEMBER: Always configure the layout of the system so that the effluent supply line to the dripper lines feeds the system from the lower elevation and layout the return field flush line from the highest elevation of the dripper field.

Other Considerations in the Layout of "Perc-Rite"™ Systems

Location of the System

The "Perc-Rite"™ System should be located in the best available soil on the lot. All setback requirements from wells, lot lines and waterways must be observed (see Figure 7 on Page 14.)

A repair area may need to be designated if required by the local health department. The exact location of the tanks and drainage or landscape improvements must be noted.

FIGURE 2

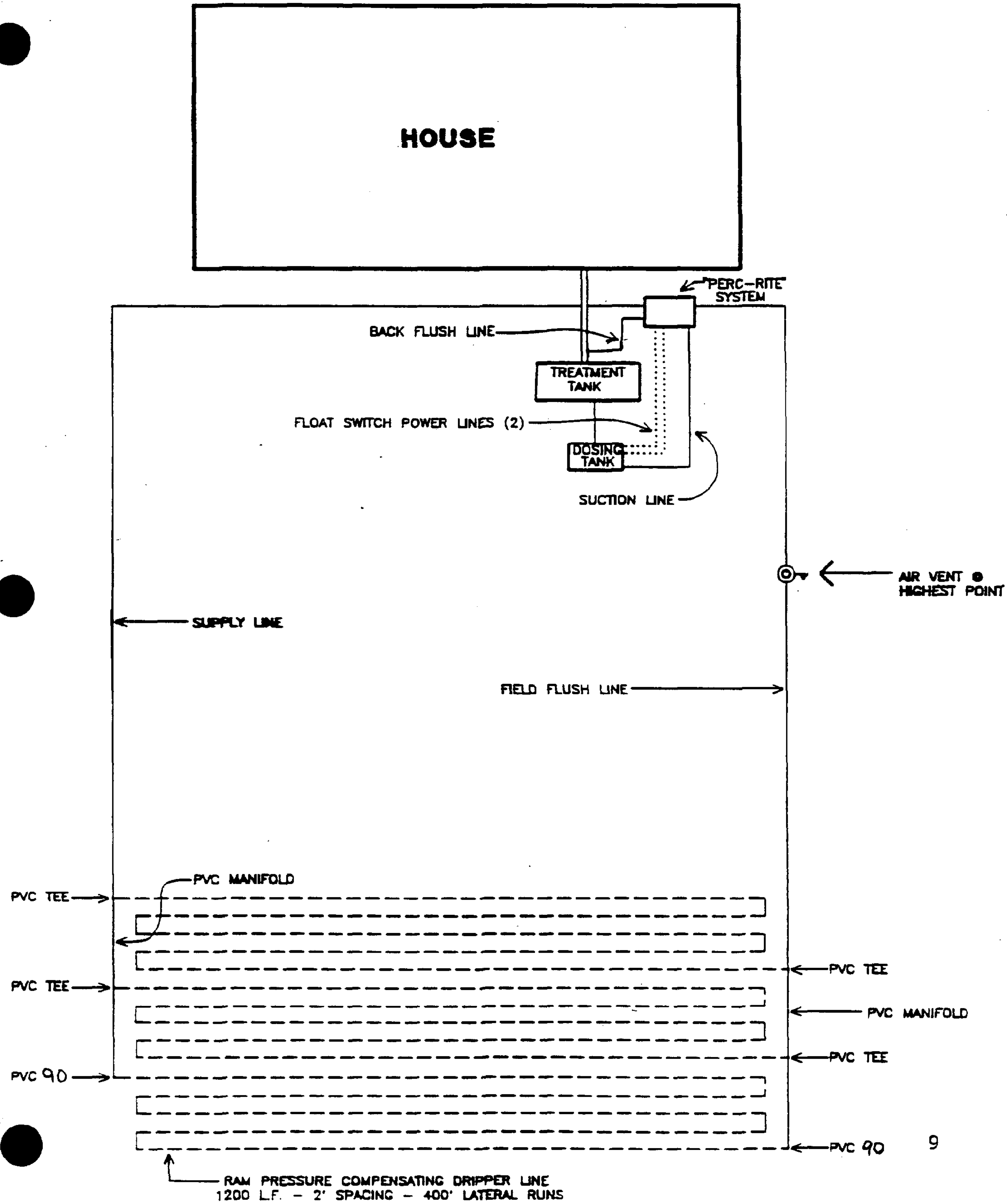

HOUSE
"PERC-RITE" SYSTEM
BACK FLUSH LINE
TREATMENT TANK
FLOAT SWITCH POWER LINES (2)
DOSING TANK
SUCTION LINE
AIR VENT @ HIGHEST POINT
SUPPLY LINE
FIELD FLUSH LINE
PVC MANIFOLD
PVC TEE
PVC TEE
PVC TEE
PVC MANIFOLD
PVC TEE
PVC 90
PVC 90
RAM PRESSURE COMPENSATING DRIPPER LINE
1200 L.F. - 2' SPACING - 400' LATERAL RUNS

FIGURE 3

COMMON RETURN AND SUPPLY

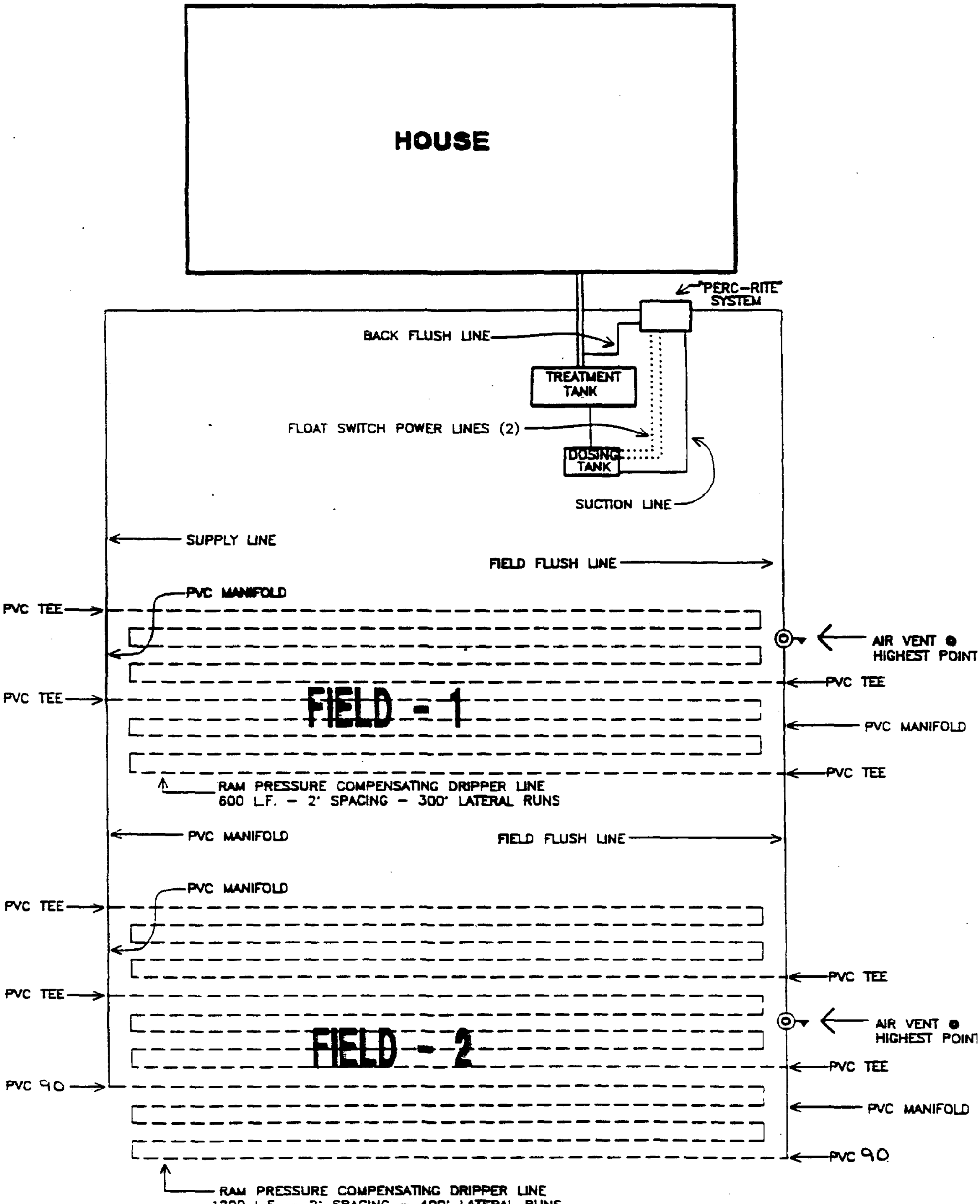
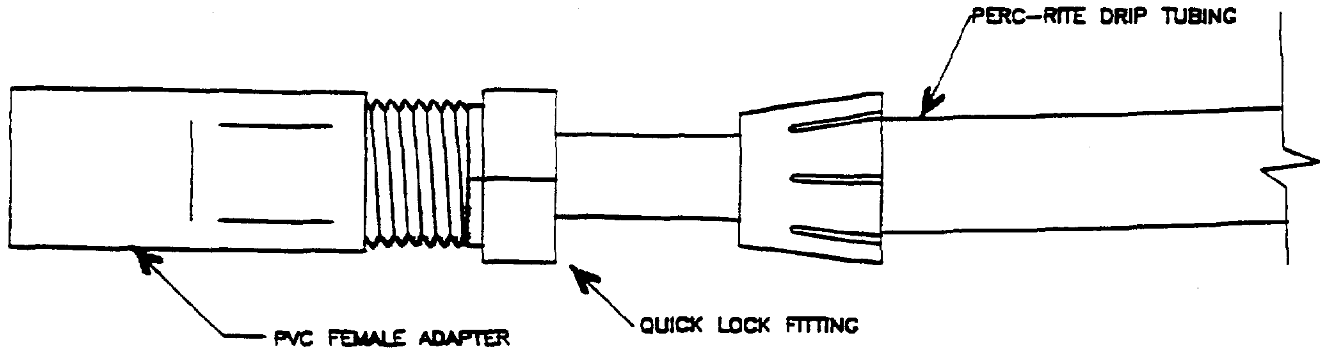
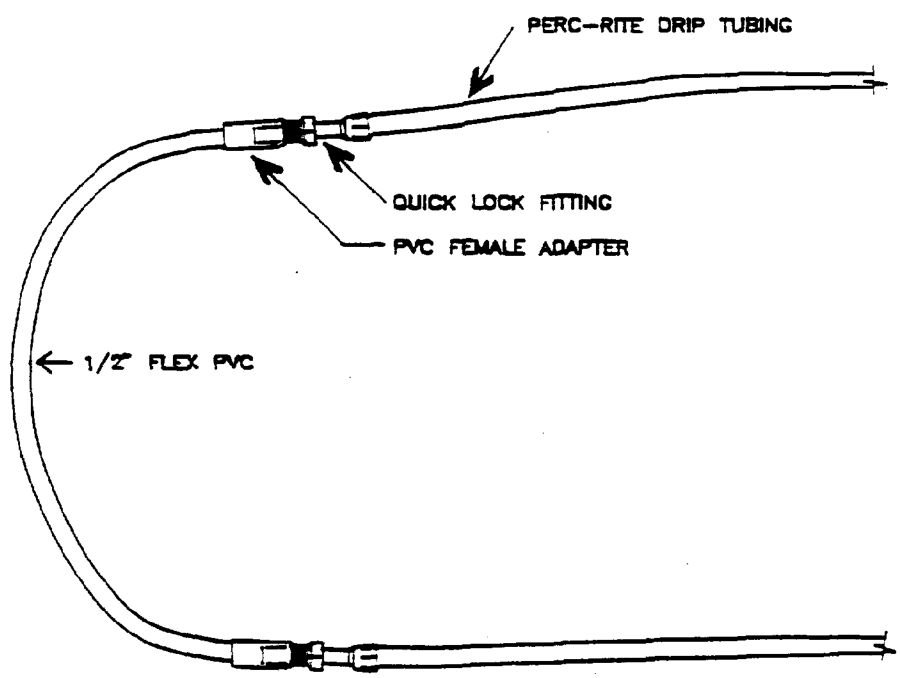


FIGURE 4

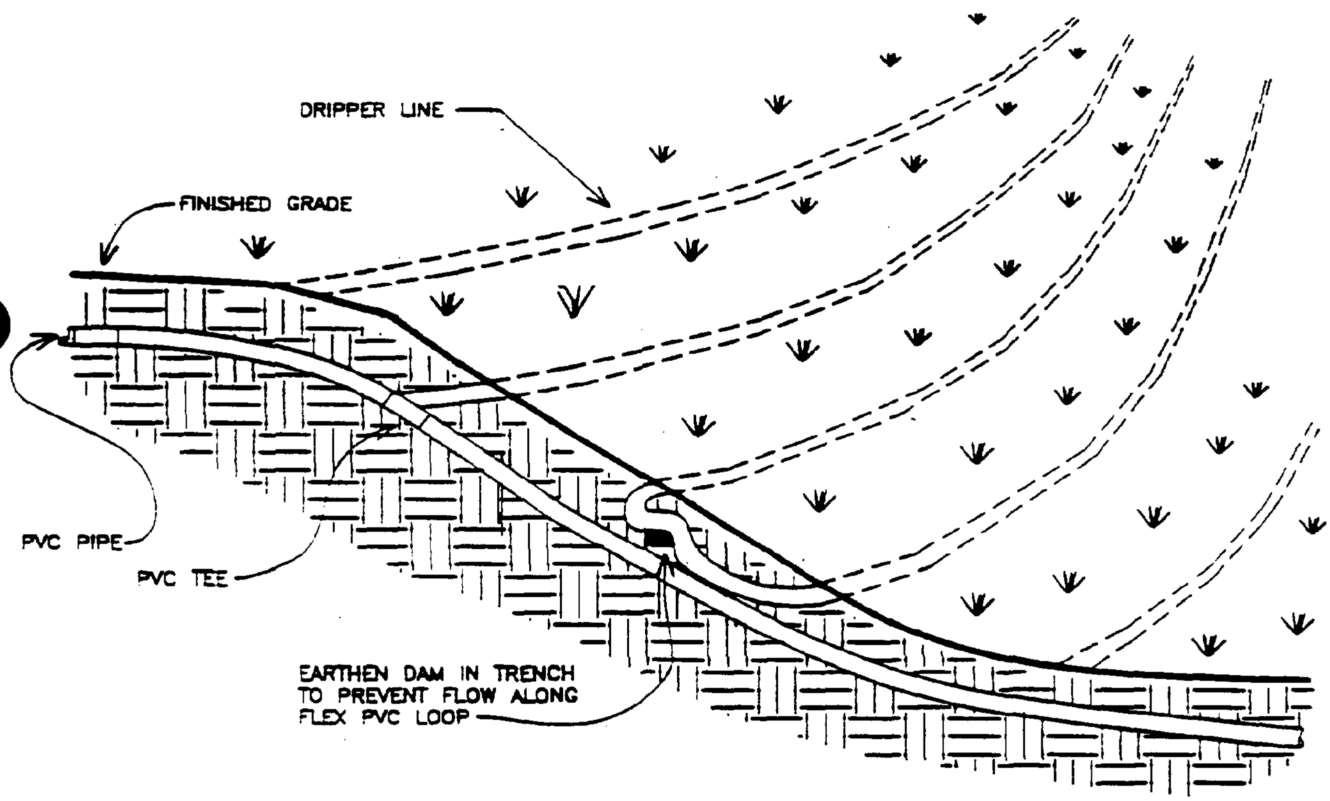


DETAIL - QUICK LOCK FITTING



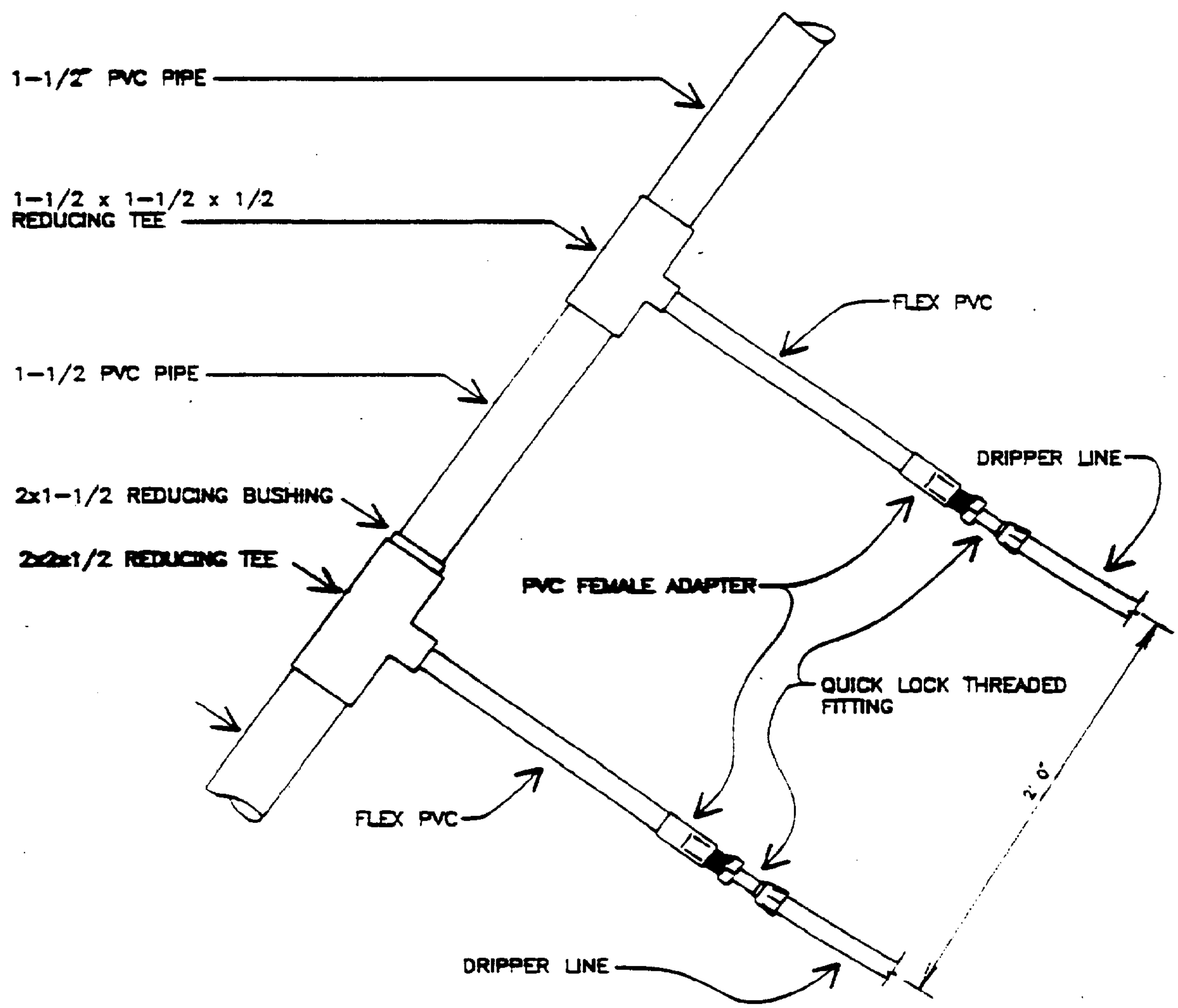
DETAIL - TYPICAL DRIPLINE CONNECTION IN TURN OR LOOP

FIGURE 5



SECTION – DRIPPER LINE TO FOLLOW CONTOURS
& PVC MANIFOLD PERPENDICULAR TO GRADE

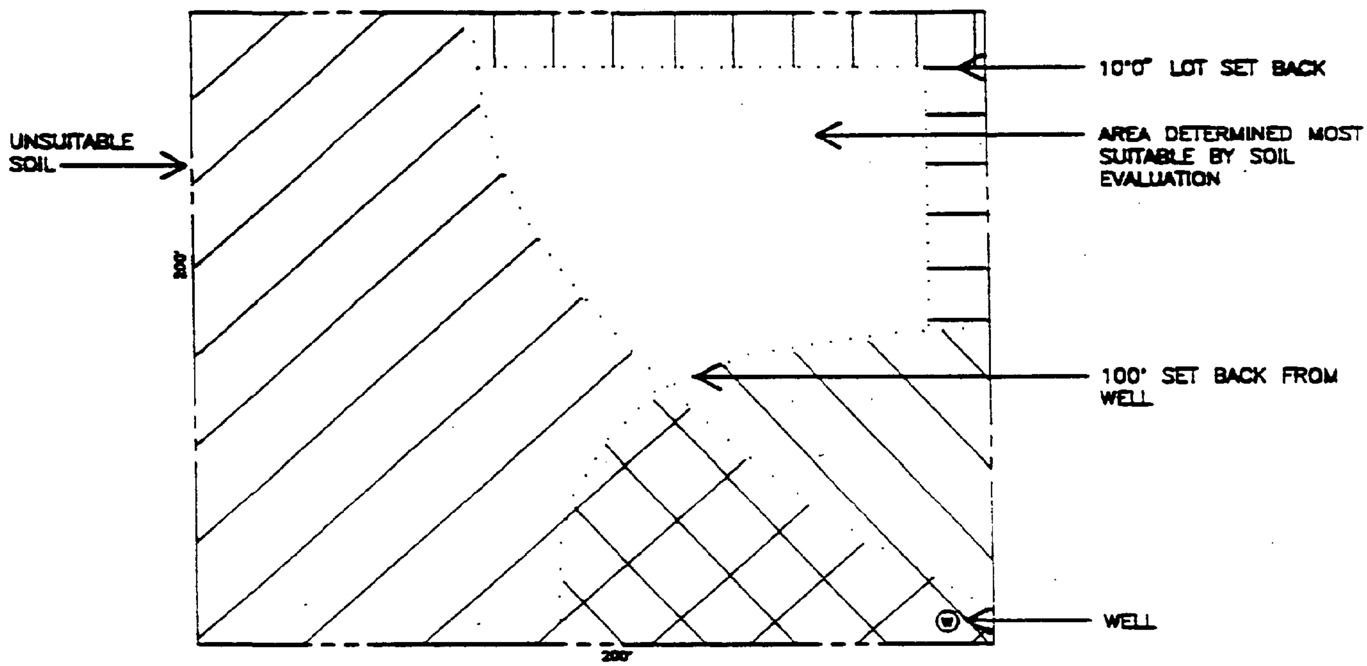
FIGURE 6



TYPICAL DRIPPERLINE CONNECTION TO PVC

(MAY VARY ACCORDING TO MANUFACTURER'S SPECIFICATIONS)

A. LOCATE SUITABLE AREAS ON SITE



B. SPECIFY LOCATION OF SYSTEM

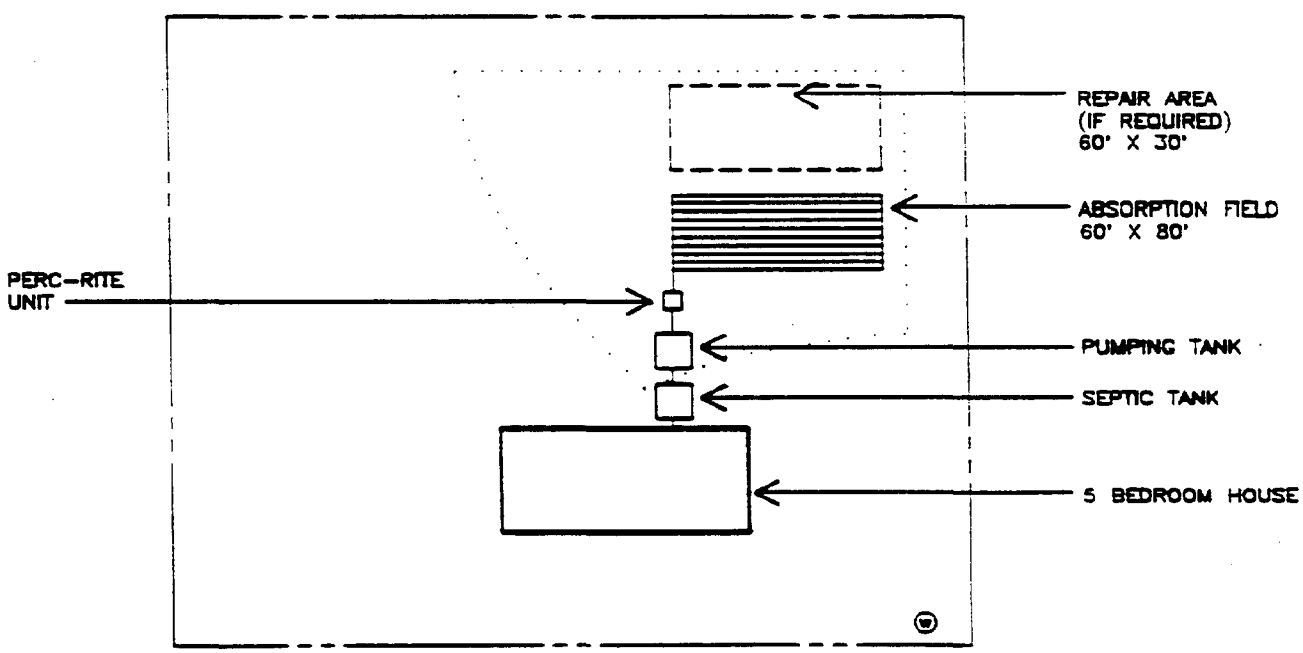


FIGURE 7

SOIL FORM

OWNER:	TELEPHONE #:	DATE:
ADDRESS:	SUBD/LOT/BLOCK:	
	COUNTY:	EST. FLOW RATE (GPD):
SOIL SCIENTIST:		
NOTES:		

TEST HOLE NO.	SOIL SERIES	SLOPE/ %	LANDSCAPE/ ASPECT	DEPTH TO GROUND WATER OR OTHER RESTRICTIVE LAYER	GULLEY DATA	EROSION	FLOOD HAZARD

SIDE ONE

TEST HOLE NO.	HORIZON	DEPTH (INCHES)	MOIST COLOR		(A) TEXTURE	(B) CONSISTENCY	PERME-ABILITY INCH/HR	PERC RATE MIN/INCH	% OF STONE	MAX HYD LOADING RATE
			MATRIX	MOTTLES						

ABBREVIATIONS:

(A) S.C.S. - SOIL: Sand - S; Loamy Sand - LS; Sandy Loam - SL; Loam - L; Sandy Clay Loam - SCL; Silt Loam - SIL; Clay Loam - CL; Silty Clay Loam - SICL; Sandy Clay - SC; Silty Clay - SIC; Clay - C.

(B) S.C.S. - CONSISTENCY: Loose - ML; Very Friable - MVFR; Friable - MFR; Firm - MFI; Very Firm - MVFI; Extremely Firm - MEFI.

SIDE TWO

Table 1 shall be used in determining the acceptance rate for drip systems. The acceptance rate shall be based on the most hydraulically limiting, naturally occurring soil horizon within two feet of the dripper line bottom.

TABLE 1

SOIL GROUP	SOIL TEXTURAL CLASSES (U.S.D.A. CLASSIFICATION)		MAXIMUM HYDRAULIC LOADING RATE
			gpd/ft^2
I	Sands (With S or PS structure)	Sand - S Loamy Sand - LS	0.4
II	Coarse Loams (With S or PS structure)	Sandy Loam - SL Loam - L	0.3
III	Fine Loams (With S or PS structure)	Sandy Clay Loam - SCL Silt Loam - SIL Clay Loam - CL Silty Clay Loam - SICL	0.15
IV	Clays (With S or PS structure)	Sandy Clay - SC Silty Clay - SIC Clay - C	0.1

Note: The acceptance rate shall be less than the maximum hydraulic loading rate for the applicable soil group for food service facilities, meat markets, and other places of business where accumulation of grease can cause premature failure of a soil absorption system. However, acceptance rates up to the maximum for the applicable soil group may be permitted for facilities where data from comparable facilities indicates that the grease and oil content of the effluent will be less than 30 mg/l and the chemical oxygen demand (C.O.D.) will be less than 250 mg/l.

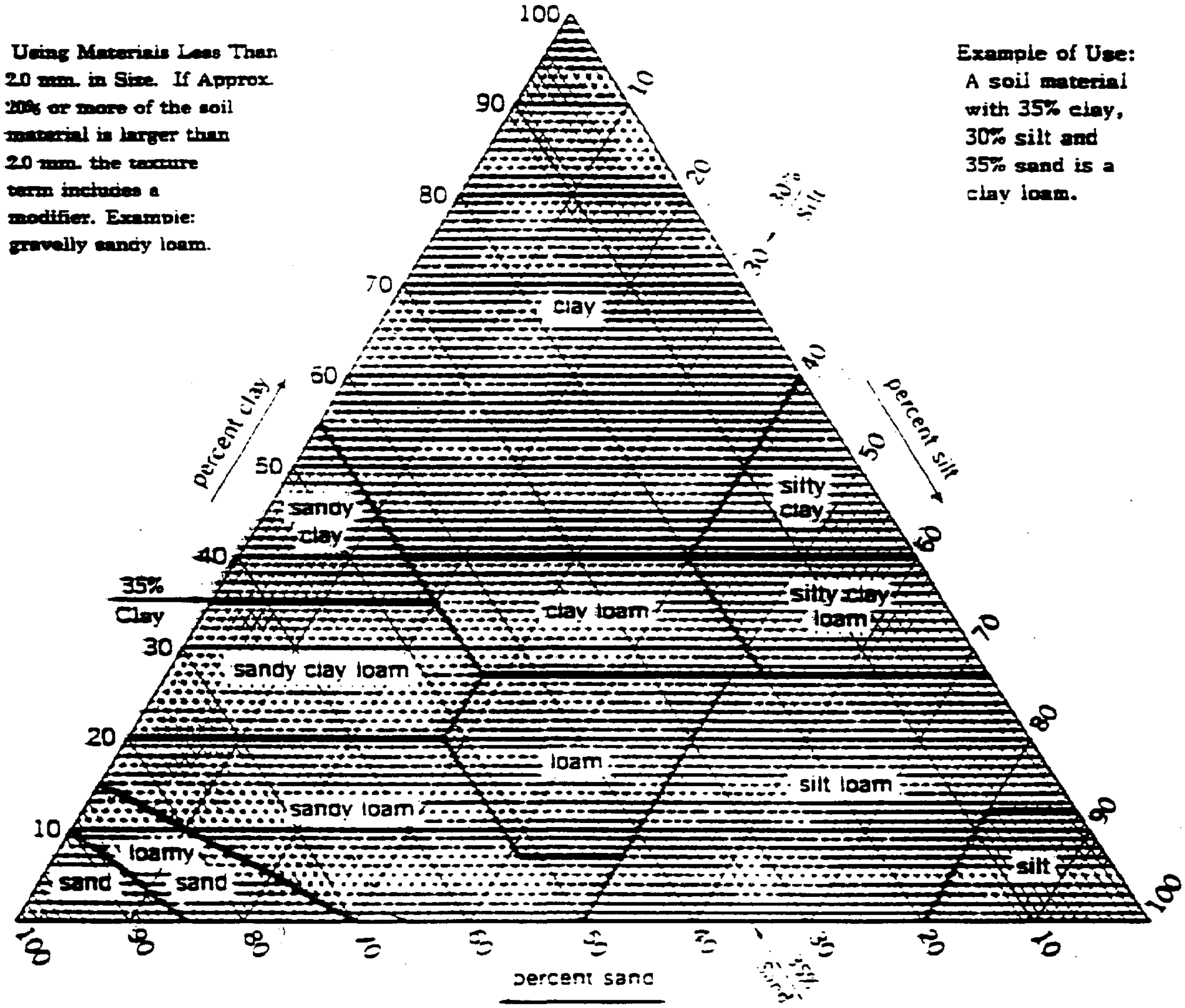
Remember: These are Maximum Rates. If textures seem inconsistant or there is adequate room, a lesser loading rate may be used.

Chart 1

CHART 1 GUIDE FOR USDA SOIL TEXTURAL CLASSIFICATION.

Using Materials Less Than 2.0 mm. in Size. If Approx. 20% or more of the soil material is larger than 2.0 mm. the texture term includes a modifier. Example: gravelly sandy loam.

Example of Use: A soil material with 35% clay, 30% silt and 35% sand is a clay loam.



EXAMPLE OF FORM FOR:

SITE EVALUATION SPECIFICATIONS

PROJECT ______________________________

OWNER ______________________________ PHONE ____________

LOCATION ______________________________

DATE ______________________________

COUNTY ______________________________

SOIL TYPE ____________________ SOIL TEXTURE ____________

SOIL GROUP ____________________ DEPTH FOR OPTIMUM USE ________

SOIL LOADING RATE ______________ PROJECTED DAILY FLOW ________

DISPOSAL AREA SIZE ______ G.P.D. ÷ ______ L/R = ______ SQ. FT.

TOTAL DRIPPER LINE (2' LATERAL SPACE) ______ SQ FT ÷ 2 = ______ L/FT

SITE SLOPE ______ % When slopes of 20% or greater are encountered use three (3) ft. minimum lateral spacing.

ABSORPTION FIELD REQUIRED PER 100 GALLONS OF WASTE WATER

LOADING RATE	SQUARE FEET	LINEAR FEET
0.4	250	125
0.3	333	167
0.15	666	333
0.1	1000	500
0.05	2000	1000

Formula For Computing Linear Footage For Other Loading Rates:

AREA REQUIRED:

$$\frac{\text{Gallons Flow/Day}}{\text{Loading Rate}} = \text{Area Required Square Feet}$$

LINEAR FEET DRIPPER LINE REQUIRED:

$$\frac{\text{Area}}{\text{Lateral Spacing}} = \text{Linear Feet}$$

Size of Septic and Pump/Dosing Tanks

Septic tank volume is determined according to state and local regulations, and is the same as a conventional system. The pumping tank should provide one day for emergency storage; therefore, twice the volume of the daily flow would definitely be sufficient.

Example: For a 450 GPD waste flow.

Volume of Pumping Tank = 450 Gallons x 2 = 900 Gallons

Aerobic treatment units (ATU), if used as the means of waste water treatment, should be sized by local health authorities and ATU manufacturer.

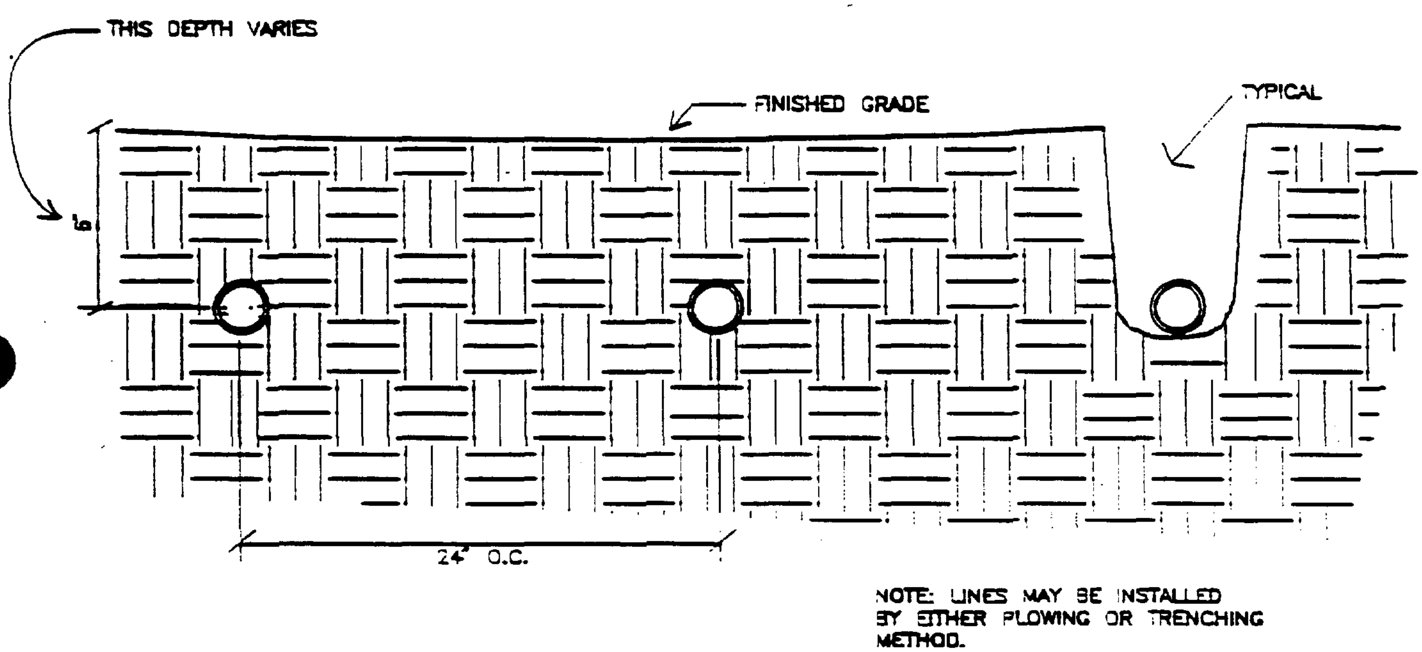
Depth of Lines

Lines may be placed anywhere in the soil profile that has been determined by the soils engineer to be the most acceptable depth. Normally, however, line depths are usually from 8 inches deep to 24 inches deep. This depth is often determined by the restrictive horizon in the soil in order to meet the separation requirement. The shallowest installation possible should be used as shown in Figure 8 (see Page 21).

Landscaping and Drainage

All landscaping, filling and site drainage completed before and after the "Perc-Rite"™ installation must be recorded and evaluated to ensure the integrity of the soil absorption system. Use of imported fill must be done in accordance with procedures described later in this manual (see Modified "Perc-Rite"™ Systems using fill in Part 6.)

FIGURE 8



IRRIGATION DRIP LINE INSTALLATION DETAIL

PART FOUR

Drip System Dosing and Design

The purpose of the "Perc-Rite"™ pressure compensating drip system dosing is to provide slow rate uniform distribution of the septic tank or ATU effluent over the entire soil absorption system. This is easily achieved using the type of dripper line incorporated in the "Perc-Rite"™ System since changes in pressure inside the tubing has little effect on the dripper emission rates. This part of the design manual will instruct you on how to ensure your "Perc-Rite"™ System components have been sized correctly.

Flow Rate during Absorption Field Dosing

The flow rate during absorption field dosing depends on the amount of dripper line required for any particular installation. The flow rates are easily calculated as follows:

The "Perc-Rite"™ RAM dripper line discharge rate is .61 gallons per emitter. This discharge rate is constant from 5 to 60 psi. (See Flow Chart - Graph 1 on Page 23.)

Distance between Emitters

In almost all cases, the distance between dripper line emitters will be a standard 24 inches. Therefore, the equation to calculate the absorption field dosing rate is as follows:

Note: GPH = Gallons per Hour
GPM = Gallons per Minute

$$\frac{\text{Dripper Line Length in Absorption Field}}{\text{2 Foot Emitter Spacing}} = \text{Number of Emitters}$$

$$\text{Number of Emitters} \times .61\ \text{GPH} = \text{Absorption Field Dosing Rate in GPH}$$

$$\frac{\text{GPH}}{\text{60 minutes}} = \text{Absorption Field Dosing Rate in GPM}$$

Example: You have 2,000 feet of dripper line required in an absorption field.

$$\frac{2{,}000 \text{ feet}}{2} = 1{,}000 \text{ emitters in the absorption field}$$

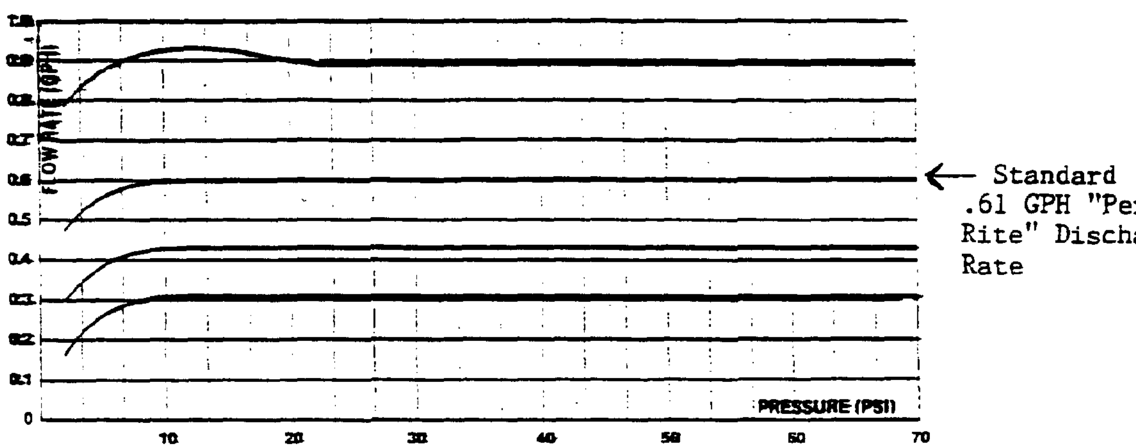
1,000 x .61 GPH = 610 GPH

$$\frac{610 \text{ GPH}}{60 \text{ minutes}} = 10.16 \text{ GPM}$$

Therefore you have a 10 gallons per minute flow in the absorption field during dosing.

Graph 1.

"Perc-Rite"™ RAM Dripper Line Discharge Rates vs. Pressure



Field Flushing Flow Rates

Since automatic flushing of the dripper lines in the absorption field is an integral function of the "Perc-Rite"™ total system, it should be considered as part of the overall flow rate generated by the system. This is important in determining if the standard "Perc-Rite"™ pump will be operating within its efficiency rating.

It has been established that proper scouring and flushing of any pipe system will require at least 1.6 gallons per minute flow at the outflow end (distal end) of any pipe. Therefore, we should require a flow of at least 1.6 gallons per minute out of each dripper line connection that has been made to the return flush manifold pipe. Therefore, multiply each return manifold connection by 1.6 GPM to get the field flushing flow requirement. This flow rate will be generated in addition to the absorption field dosing flow rate.

Example: Assume you have 2,000 feet of dripper line required in the absorption field. The field configuration and layout has allowed for 400 feet (maximum) lateral dripper line runs or loops before connection to the field flush manifold line. Therefore, there will be five connections to the field flush line.

Equation to determine the field flush flow requirement:

5 connections x 1.6 GPM = 8 GPM

So 8 GPM is required.

Note: Regardless of field layout or configuration, use at least 1.6 GPM multiplied by the number of return flush line connections.

Example:

$$\frac{\text{1,800 linear feet of dripper line}}{\text{200 feet lateral runs}} = \text{9 manifold connections to flush line}$$

9 x 1.6 GPM = 14.4 GPM Flushing Flow

Total Flow Requirement of System

The total flow used in calculating the operating flow requirement of the "Perc-Rite"™ absorption field would be the combination of both the field dosing flow and the field flushing flow.

Example:

$$\frac{\text{2,000 L.F. Drip}}{2} = \text{1,000 Emitters}$$

1,000 Emitters x .61 GPH = 610 GPH

$$\frac{\text{610 GPH}}{\text{60 Minutes}} = \text{10.1 GPM}$$

Therefore, you use 10 GPM.

2,000 L.F. Drip
--------------- = 5 Connections
400 Foot Runs

5 connections x 1.6 GPM = 8 GPM

Total Flow Requirement = 10 GPM + 8 GPM = 18 GPM

Therefore, 18 GPM is the Total Flow Requirement in this example.

Pump Size Verification

To determine if the "Perc-Rite"™ System pump is the correct size, flow friction and head losses must be considered. Once the flow friction and head losses have been calculated and an operating pressure requirement has been established, this can be compared to the pump performance chart for the standard "Perc-Rite"™ unit. This will determine if the system is designed within the operating capabilities of that standard system. If the system as designed exceeds the performance characteristics of the standard "Perc-Rite"™ unit, you may then modify the system design to work within the "Perc-Rite"™ operating capabilities. Such as splitting the absorption field into two or more separate zones that operate independently, change pipe sizes, dripper line lateral lengths or any combination of these items. Changing the pump size to larger than standard is also an option. All of these options will be discussed in this section.

Calculating Flow Losses and Requirements for "Perc-Rite"™ W-20 Systems

Friction and Head Losses to Consider

A. Suction Line Loss
B. Suction Lift
C. Pump Discharge Supply Line to Absorption Field-Force Main
D. Return Flush Line Loss
E. "Perc-Rite"™ Pump and Filter Unit Loss
F. Pipe Fittings
G. Elevation Changes
H. Dripper Line Laterals

Calculation Methods

A. SUCTION LINE LOSS -

Using Chart 1A, calculate the friction loss in your suction line from dosing tank to pump unit. 1 1/4" PVC schedule 40 is a standard for the "Perc-Rite"™ suction line.

How to use the chart. After calculating the total flow requirement of your absorption field as discussed previously, find that flow in GPM in the first column of Chart 1A (see Page 34). Follow the line that corresponds to the flow requirement to the column under the pipe size at the top of Chart 1A (see Page 34) corresponding to your suction line size (remember 1 1/4" is a standard). However, it is possible, due to system size, that a larger pipe size will make the system design more efficient. Find the loss number in that column for your design flow. This loss is in feet of head. Remember this head loss will be per 100 feet of pipe. In order to convert this head loss into psi pressure loss, divide this number by 2.31. The 2.31 value is the conversion factor for finding pressure loss (in psi) out of the head loss in your pipe. Therefore, the equation for finding your pressure loss in the suction line is as follows:

$$\frac{\text{Head Loss in Suction Line from Chart 1A}}{2.31} = \text{PSI Loss}$$

Note: Always use the next higher number in the chart if your calculated flow does not exactly match the chart flow in GPM. (Example: Suppose the design flow is 14 GPM. 14 GPM is not on the chart, so use 15 GPM, which is on the chart, as the flow.)

Example: System absorption field total flow is 18 GPM (as calculated in the previous example of finding operating flow.)
Chart 1A shows 18 GPM in 1 1/4" pipe to lose 4.28 in head loss per 100 feet.

Assume a 40 foot suction line length then loss is:

$$4.28 \times .4 = 1.71$$

Convert the head loss to psi: $\frac{1.71}{2.31} = .74$ psi

To be safe you may round to 1 psi; therefore, suction line loss is 1 psi.

B. SUCTION LIFT -

Check suction lift by elevation change from suction line intake to the pump location. This number should be used when checking the pump performance in Chart 4A (see Page 37). Pump performance charts show pumping depth in five foot increments. Round the actual elevation difference up to the nearest multiple of five when checking pump performance.

Example: Intake of the suction line is 8 vertical feet lower than the elevation of the "Perc-Rite"™ pump unit location. Round the 8 feet up to the nearest multiple of five or 10 feet.

When checking pump performance, Chart 4A (see Page 34) as discussed later, use 10 feet as suction lift.

C. PUMP SUPPLY LINE TO ABSORPTION FIELD - FORCE MAIN

This calculation will be the same as described for suction line loss. You will again use Chart 1A. The length of the supply line from the "Perc-Rite"™ pump unit to the farthest point in the absorption field area should be used in calculating this flow loss. You will again use the total flow requirement of the absorption field in GPM which is the first column in Chart 1A. Find the column which corresponds to the supply line size intended. Use the loss column for that size pipe at the absorption field total flow rate. The conversion to pressure loss in psi will be the same as in the suction loss.

$$\frac{\text{Head Loss}}{2.31} = \text{PSI Loss}$$

Example: Again using 18 GPM total flow, assume 1 1/4" supply pipe, Chart 1A shows 18 GPM in 100 feet of 1 1/4" pipe is 4.28 head loss. The supply line is 150 feet to the farthest point in the system. The loss is 4.28 x 1.5 = 6.42. Then convert to psi.

$$\frac{6.42 \text{ head loss}}{2.31} = 2.78 \text{ PSI Loss}$$

The supply line to field loss is 2.78 psi.

D. RETURN FLUSH LINE LOSS -

The return flush line loss is figured in the same manner as the absorption field supply line loss as discussed in Part C. Always remember to use a return flush pipe size equal to the supply line size. Use as your length of line for this calculation the total length of pipe from the "Perc-Rite"™ pump/filter unit to the farthest point at which a dripper line lateral connects to the return flush line.

Example: The flush line at the farthest point away from the "Perc-Rite"™ pump/filter unit is 180 feet. The supply line to the absorption field is 1 1/4", so the return flush pipe will be the same size or 1 1/4".

Note: As discussed earlier, proper flushing velocities are calculated at 1.6 gallons per minute per dripper lateral flush line connection. Therefore, to calculate the flow used to determine loss in return flush lines multiply 1.6 x the number of flush line connections.

The earlier example of total flow of 18 GPM was for a 2,000 linear feet absorption field with 400 feet lateral dripper line runs. Therefore, we will use five connections to the return flush line in this example.

Example: 5 connections x 1.6 GPM = 8 GPM

Use 8 GPM as flushing flow through the return line.

Now use Chart 1A again to calculate losses through the flush line. Chart 1A shows 8 GPM through 1 1/4" pipe is .95 head loss. Suction line is 180 feet long. Therefore:

$$1.8 \times .95 = 1.71 \qquad \frac{1.71}{2.31} = .74 \text{ psi (Round up to 1)}$$

Loss in return flush line is 1 psi.

E. "PERC-RITE"™ PUMP/FILTER UNIT LOSSES -

All the figures for pressure loss in the valves, flow meter, filters, fittings, etc. of the "Perc-Rite"™ preassembled pump/filter unit have been calculated and are found in Chart 2A ("Perc-Rite"™ Filter Unit Losses, see Page 35). In order to find the flow loss through the "Perc-Rite"™ Pump/Filter Unit, you will use the total flow rate calculated for the absorption field and find that flow rate in the first column of Chart 2A. The second column will give you the flow loss in psi through the unit.

Example: Again use 18 GPM, as calculated previously. The chart shows at 18 GPM pressure loss will be 10.35 psi. Therefore, the "Perc-Rite"™ Pump/Filter Unit Loss would be 10.35 psi in this example.

F. PIPE FITTINGS -

During all calculations in determining pressure/head losses and requirements, you generally have been rounding up to the nearest whole number or as in the case of the suction lift, you always round up to the nearest multiple of five (as discussed in Part B). Therefore, the insignificant pressure loss through fittings in the "Perc-Rite"™ W-20 residential system will not require any calculations and will not effect operation of the system as discussed in this section. Use as few fittings as possible.

G. ELEVATION CHANGES -

Changes in elevation will contribute to the total losses that need to be considered. The area of the "Perc-Rite"™ System which must be considered for losses due to elevation changes is:

The elevation difference from the "Perc-Rite™ Pump Unit to the highest point in the absorption field.

Remember: As discussed earlier, you will always design the supply line to feed the drip absorption field from the <u>lowest</u> elevation of the absorption field and the return flush line feeds from the highest elevation in the absorption field.

Converting elevation changes to pressure losses in psi. It is known that for every 2.31 feet in elevation rise the pressure required to overcome that elevation change is 1 psi; therefore, the equation to convert elevation change to pressure loss is:

$$\frac{\text{Number of Feet in Elevation Rise}}{2.31} = \text{PSI Loss}$$

To find pressure loss in elevation changes, first determine the change in elevation <u>rise</u>, if any, from pump/filter unit location to the highest point in the absorption field which most likely will be at the highest flush line connection.

Example #1: The pressure loss computation is as follows:

The elevation rise is 15 vertical feet from the "Perc-Rite"™ Unit to the highest point in the absorption field.

$$\frac{\text{15 feet Elevation Rise}}{2.31} = 6.49 \text{ (Round to 6.5 PSI)}$$

The pressure loss in the elevation rise is 6.5 psi.

Example #2: The pressure loss computation is as follows:

The elevation rise is 0 vertical feet, or the field is approximately the same elevation as the "Perc-Rite"™ pump unit.

$$\frac{0 \text{ Elevation Rise}}{2.31} = 0 \text{ PSI Loss}$$

The pressure loss in the elevation rise is 0 psi.

H. DRIPPER LINE LATERALS -

Pressure/friction losses for the dripper line laterals or loops must be considered in the overall pressure requirements of the pump system. This pressure is usually the most significant amount of pressure losses to consider.

The calculation of the pressure requirement has been made simple by the use of Chart 3A ("Perc-Rite"™ Drip Line Pressure Requirements on Page 36). The pressure requirements for maintaining the minimum required 5 psi operating pressure throughout the dripper line lateral has already been calculated and put on the chart for varied lengths of dripper line this chart has also taken into consideration the pressure required to maintain the 1.6 GPM minimum flushing velocities in the dripper line lateral and friction loss.

Simply determine the length of the longest dripper line lateral or loop in your system design and refer to Chart 3A (see Page 36). Find the corresponding inlet pressure requirement for the longest lateral length in your system. This pressure requirement will then be the minimum required pressure to maintain correct dripper line operation and flushing throughout the entire absorption field since the longest lateral or loop was used in determining that pressure.

Example: An absorption field requiring 2,000 feet of dripper line and having five lateral lines or loops. The longest lateral is 400 feet. Chart 3A shows an inlet pressure requirement corresponding to 400 feet lateral length is 25.4 psi.

The pressure requirement consideration for dripper line laterals is 25.4 psi in this example.

<u>Summary of Calculations for Pump Size Verification Procedure</u>

The first step in any design verification will have been the determination of the absorption field size, based on flow and soil loading rates. Then pump sizing verification will be as follows:

Determine Absorption Field Total Operating Flow

$$\frac{2{,}000 \text{ Linear Feet}}{2 \text{ Foot Spacing}} = 1{,}000 \text{ Emitters}$$

1,000 Emitters x .61 GPH Rate = 610 GPH

$$\frac{610 \text{ GPH}}{60 \text{ Minutes}} = 10.1 \text{ GPM}$$

Round off to 10 Gallons per Minute

5 Lateral Lines 400' Long x 1.6 GPM Flushing Velocity = 8 GPM

10 GPM Operating + 8 GPM Flushing = 18 GPM

18 GPM Total Operating Flow

Note: Flushing flow will be an important separate flow rate as discussed later in system modifications.

Determine System Head and Friction Loss

A. Suction Line - Use Chart 1A and 2.31 as a conversion factor.

$$1\ 1/4\text{" Pipe at 18 GPM} = 4.28 \times \frac{40 \text{ Feet}}{100 \text{ Feet}} = 1.71$$

$$\frac{1.71}{2.31} = .74 \text{ psi} \quad \text{(Round off to 1 psi)}$$

B. Suction Lift - Determine vertical lift from the suction intake to pump location - compare in pump Chart 5A when verifying the pump size.

C. Force Main - Again, use Chart 1A.

1 1/4" Pipe at 18 GPM = $4.28 \times \frac{150}{100} = 6.42$

$\frac{6.42}{2.31} = 2.78$ psi

D. Return Flush - Again use Chart 1A.

5 Connections to flush line x 1.6 GPM = 8 GPM

1 1/4" Pipe at 8 GPM = $\frac{.95 \text{ (Use 1)}}{2.31} = .43$ psi (Use .5 psi)

E. "Perc-Rite"™ Pump/Filter Unit - Use Chart 2A.

18 GPM shows 10.35 psi.

F. Pipe Fittings - No calculations required for "Perc-Rite"™ Residential Units.

G. Elevation Changes -

$$\frac{\text{Number of Feet in Elevation Rise}}{2.31} = \text{PSI Loss}$$

Elevation rise = 2 feet

Total Elevation Loss = $\frac{2 \text{ Feet}}{2.31} = .86$ (Round to 1 PSI)

H. Dripper Line Laterals - Use Chart 3A.

The longest dripper line lateral is 400 feet. Chart 3A shows 25.4 psi requirement corresponding to 400 feet lateral length. (Use 25.4 psi)

Calculate Total Pressure Loss Considerations

A.	Suction Line -	1.00 psi
B.	Suction Lift - To be used in pump Chart 4A	
C.	Supply Line -	2.78 psi
D.	Return Flush -	1.00 psi
E.	Pump Filter Unit -	10.35 psi
F.	Pipe Fittings -	0.00 psi
G.	Elevation Changes -	1.00 psi
H.	Dripper Line Req. -	25.40 psi

Total Pressure Required (Add A-H together) 41.53 psi

Friction loss in plastic pipe - Schedule 40

CHART 1A.

Velocity measured in ft./sec., Loss in feet of water head per 100 ft. of pipe.

GALS. PER	½"		¾"		1"		1¼"		1½"		2"		2½"		3"		3½"		4"	
MIN.	Vel	Loss	Vel	Loss	Vel	Loss	Vel	Loss	Vel	Loss	Vel	Loss	Vel	Loss	Vel	Loss	Vel	Loss	Vel	Loss
2	2.10	3.47	1.20	0.89																
4	4.23	12.7	2.41	3.29	1.49	1.01	.86	.27	.63	.12										
6	6.34	26.8	3.61	6.91	2.23	2.14	1.29	.57	.94	.26	.57	.09								
8	8.45	46.1	4.82	11.8	2.98	3.68	1.72	.95	1.26	.45	.77	.16	.52	.05						
10	10.6	69.1	6.02	17.9	3.72	5.50	2.14	1.44	1.57	.67	.96	.24	.65	.08	.43	.03				
12			7.22	24.9	4.46	7.71	2.57	2.02	1.89	.94	1.15	.37	.78	.11	.52	.05				
15			9.02	37.6	5.60	11.8	3.21	3.05	2.36	1.41	1.50	.51	.98	.17	.65	.07	.49	.03		
18			10.8	50.9	6.69	16.5	3.86	4.28	2.83	1.99	1.72	.70	1.18	.24	.78	.10	.58	.04		
20	5" PIPE		12.0	63.9	7.44	19.7	4.29	5.21	3.15	2.44	1.01	.86	1.31	.29	.87	.12	.65	.05	.51	.03
25					9.30	30.1	5.36	7.80	3.80	3.43	2.50	1.28	1.63	.43	1.09	.18	.81	.08	.64	.04
30	.49	.02			11.15	41.8	6.43	10.8	4.72	5.17	2.89	1.80	1.96	.61	1.30	.25	.97	.11	.77	.06
35	.57	.03			13.02	55.9	7.51	14.7	5.51	6.91	3.35	2.40	2.35	.81	1.52	.33	1.14	.15	.89	.08
40	.65	.04	6" PIPE		14.88	71.4	8.58	18.8	6.30	8.83	3.82	3.10	2.68	1.03	1.74	.43	1.30	.19	1.02	.10
45	.73	.04			16.70		9.65	23.5	7.08	10.9	4.30	3.85	3.02	1.32	1.95	.54	1.46	.24	1.15	.13
50	.82	.05	.57	.02			10.72	28.2	7.87	13.3	4.78	4.65	3.35	1.56	2.17	.65	1.62	.29	1.28	.16
55	.90	.06	.62	.02			11.78	33.8	8.66	16.0	5.26	5.55	3.69	1.88	2.39	.74	1.70	.34	1.41	.19
60	.98	.07	.68	.03			12.87	40.0	9.44	15.6	5.74	6.53	4.02	2.19	2.60	.90	1.95	.40	1.53	.22
65	1.06	.09	.74	.04			13.92	46.7	10.23	21.6	6.21	7.56	4.36	2.53	2.82	1.02	2.00	.47	1.66	.25
70	1.14	.10	.79	.04			15.01	53.1	11.02	24.9	6.69	8.64	4.69	2.91	3.04	1.21	2.27	.54	1.79	.30
75	1.22	.11	.85	.05			16.06	60.6	11.80	28.2	7.17	9.82	5.05	3.33	3.25	1.41	2.32	.60	1.91	.34
80	1.31	.13	.91	.05			17.16	68.2	12.69	32.0	7.65	11.1	5.36	3.71	3.49	1.54	2.60	.69	2.04	.38
85	1.39	.15	.96	.06			18.21	77.0	13.38	35.3	8.13	12.5	5.70	3.81	3.69	1.66	2.62	.76	2.17	.42
90	1.47	.16	1.02	.07			19.30	84.6	14.71	39.5	8.61	13.8	6.03	4.61	3.91	1.92	2.92	.85	2.30	.47
95	1.55	.18	1.08	.07					14.95	43.7	9.08	15.3	6.37	5.07	4.12	2.04	2.93	.96	2.42	.53
100	1.63	.19	1.13	.08					15.74	47.9	9.56	16.8	6.70	5.64	4.34	2.33	3.25	1.03	2.55	.57
110	1.79	.23	1.25	.10					17.31	57.3	10.5	20.2	7.37	6.81	4.77	2.82	3.57	1.25	2.81	.69
120	1.96	.27	1.36	.11	8" PIPE				18.89	67.2	11.5	23.5	8.04	7.89	5.21	3.29	3.99	1.45	3.06	.80
130	2.12	.31	1.47	.13					20.46	78.0	12.4	27.3	8.71	8.79	5.64	3.81	4.22	1.68	3.31	.93
140	2.29	.36	1.59	.15	.90	.04			22.04	89.3	13.4	31.5	9.38	10.5	6.08	4.32	4.54	1.93	3.57	1.07
150	2.45	.41	1.70	.17	.96	.04			23.6		14.3	35.7	10.00	12.0	6.51	4.93	4.87	2.19	3.82	1.23
160	2.61	.46	1.80	.19	1.02	.05					15.3	40.4	10.7	13.6	6.94	5.54	5.19	2.47	4.08	1.37
170	2.77	.51	1.92	.21	1.08	.05					16.3	45.1	11.4	16.0	7.36	6.25	5.52	2.75	4.33	1.53
180	2.94	.57	2.04	.24	1.15	.06					17.2	50.3	12.1	16.8	7.81	6.58	5.85	3.07	4.60	1.70
190	3.10	.63	2.16	.26	1.21	.07	10" PIPE				18.2	55.5	12.7	18.6	8.24	7.28	6.17	3.39	4.84	1.88
200	3.27	.70	2.27	.29	1.28	.07					19.1	60.6	13.4	20.3	8.68	8.36	6.50	3.73	5.11	2.06
220	3.59	.83	2.44	.34	1.40	.08	.90	.03			21.0	72.4	14.7	24.9	9.55	10.0	7.14	4.45	5.62	2.44
240	3.92	.98	2.67	.41	1.53	.10	.98	.03			22.9	85.5	16.1	28.7	10.4	11.8	7.79	5.22	6.13	2.91
260	4.25	1.13	2.89	.47	1.66	.12	1.06	.04			24.9	99.2	17.4	33.0	11.3	13.7	8.44	6.07	6.64	3.28
280	4.50	1.30	3.11	.54	1.79	.13	1.15	.04					18.8	38.1	12.2	15.7	9.09	6.95	7.15	3.85
300	4.90	1.48	3.33	.62	1.91	.15	1.22	.05					20.1	43.2	13.0	17.9	9.74	7.90	7.66	4.37
320	5.13	1.66	3.56	.69	2.05	.17	1.31	.06					21.6	48.4	13.9	20.1	10.40	8.88	8.17	4.93
340	5.44	1.87	3.78	.76	2.18	.19	1.39	.07	12" PIPE				22.9	54.5	14.8	22.5	11.00	9.96	8.58	5.50
360	5.77	2.07	4.00	.86	2.30	.21	1.47	.07					24.2	60.2	15.6	24.9	11.70	11.0	9.10	6.15
380	6.19	2.28	4.22	.94	2.43	.24	1.55	.08	1.08	.03			25.6	66.7	16.5	27.7	12.3	12.2	9.59	6.58
400	6.44	2.5	4.43	1.03	2.60	.25	1.63	.09	1.14	.04			26.8	73.3	17.4	30.6	13.0	13.4	10.10	7.52
450	7.20	3.1	5.00	1.29	2.92	.32	1.84	.11	1.28	.05					19.5	36.7	13.9	16.7	11.49	9.31
500	8.02	3.8	5.56	1.36	3.19	.39	2.04	.13	1.42	.05					21.7	46.1	16.2	20.3	12.6	11.3
550	8.82	4.5	6.11	1.86	3.52	.46	2.24	.16	1.56	.06					23.9	55.0	17.9	24.3	13.0	13.5
600	9.62	5.3	6.65	2.19	3.85	.54	2.45	.18	1.70	.07					26.0	64.4	19.5	28.5	15.10	15.8
650	10.40	6.2	7.22	2.53	4.16	.63	2.65	.21	1.84	.09					28.2		21.1	33.0	16.40	18.3
700	11.2	7.1	7.78	2.92	4.46	.72	2.86	.24	1.99	.10							22.7	37.9	17.60	21.1
750	12.0	8.1	8.34	3.35	4.80	.82	3.06	.28	2.13	.11							24.4	43.0	18.90	24.0
800	12.8	9.1	8.90	3.74	5.10	.89	3.26	.31	2.27	.13							26.0	48.4	20.20	26.8
850	13.6	10.2	9.45	4.21	5.48	1.03	3.47	.35	2.41	.15							27.6	54.1	21.4	30.1
900	14.4	11.3	10.0	4.75	5.75	1.16	3.67	.39	2.56	.16									22.7	33.4
950	15.2	12.5	10.5	5.26	6.06	1.35	3.88	.43	2.70	.18										
1000	16.0	13.7	11.1	5.66	6.38	1.40	4.08	.48	2.84	.19										
1100	17.6	16.4	12.2	6.84	7.03	1.65	4.49	.56	3.13	.23										
1200	19.61	19.2	13.3	8.04	7.66	1.96	4.90	.66	3.41	.27										
1300	20.8		14.4	8.6	8.30	2.28	5.31	.76	3.69	.31										
1400	22.4		15.6	10.6	8.95	2.59	5.71	.88	3.98	.37										
1500	24.0		16.7	12.0	9.58	2.93	6.12	1.00	4.26	.42										
1600	25.6		17.8	12.6	10.21	3.29	6.53	1.12	4.55	.46										
1800			20.0		11.50	4.13	7.35	1.39	5.11	.57										
2000			22.2		12.78	5.03	8.16	1.69	5.68	.70										
2200			24.4		14.05	6.00	8.98	1.99	6.25	.85										
2400			26.7		15.32	6.7	9.80	2.37	6.81	.98										
2600							10.61	2.73	7.38	1.14										
2800							11.41	3.15	7.95	1.29										
3000							12.24	3.58	8.52	1.48										
3200							13.05	3.7	9.10	1.65										
3500							14.30	4.74	9.95	1.96										
3800							15.51	6.3	10.80	2.30										
4200									11.92	2.76										
4500									12.78	3.24										
5000									14.20	3.95										
5500																				
6000																				

*Data shown is calculated from Williams and Hazen formula $H = \frac{3.023}{C^{1.852}} \frac{V^{1.852}}{D^{1.167}}$ using C-150. For water at 60°F. Where H = head loss, V = fluid velocity ft./sec., D = diameter of pipe, ft.

C = coefficient representing roughness of pipe interior surface.

CHART 2A.

"PERC-RITE"™ FILTER UNIT LOSSES

TOTAL ABSORPTION FIELD FLOW GPM	TOTAL LOSS IN PSI
5	2.00
6	2.00
7	2.00
8	2.50
9	3.00
10	3.45
11	3.95
12	4.80
13	5.08
14	5.75
15	6.85
16	8.40
17	9.40
18	10.35
19	11.80
20	12.25
21	14.15
22	15.30
23	17.00
24	18.35
25	20.25

CHART 3A.

"PERC-RITE"™ DRIPPER LINE PRESSURE REQUIREMENTS
(Shown in 10 Feet Increments - Round to Nearest Multiple of Ten)

DRIPPER LINE LATERAL LGTH	PRESSURE PSI REQUIREMENTS	DRIPPER LINE LATERAL LGTH	PRESSURE PSI REQUIREMENTS
50 FEET	7.0	280 FEET	13.5
60	7.0	290	14.3
70	7.0	300	15.2
80	7.0	310	16.0
90	7.0	320	16.9
100	7.0	330	17.9
110	7.0	340	18.9
120	7.0	350	19.9
130	7.0	360	20.9
140	7.0	370	22.0
150	7.0	380	23.1
160	7.0	390	24.2
170	7.0	400	25.4
180	7.0	410	26.6
190	7.4	420	27.9
200	8.0	430	29.2
210	8.6	440	30.5
220	9.2	450	31.9
230	9.9	460	33.3
240	10.5	470	34.7
250	11.2	480	36.2
260	12.0	490	37.7
270	12.7	500	39.3

CHART 4A.
PUMP PERFORMANCE CHARTS

PUMP MODEL NUMBER	MOTOR HORSE-POWER	DISCHARGE PRESSURE PSI	SUCTION LIFT & PUMP CAPACITY (GPM)				
			5'	10'	15'	20'	25'
5 HS	1/2	30	15.25	13.5	12.0	9.5	7.4
		40	14.4	12.8	11.75	9.4	7.3
		50	9.0	8.3	7.0	6.0	5.5
OJS-50 STD. W-20 PUMP	1/2	20	17.7	14.8	12.7	9.8	7.3
		30	16.4	14.5	12.7	9.8	7.3
		40	9.6	8.8	8.0	6.8	5.3
		50	4.7	3.8	2.8	1.5	---
OJS-75	3/4	20	21.5	19.0	17.1	12.8	9.1
		30	21.5	18.8	17.1	12.8	9.1
		40	15.2	13.8	12.8	10.9	8.5
		50	8.0	6.6	5.6	3.5	1.1
OJS-100	1	20	23.4	20.3	18.8	14.3	10.7
		30	23.4	20.1	18.7	14.3	10.7
		40	20.8	15.8	14.8	12.4	9.4
		50	10.3	7.8	6.8	4.1	1.3
JS-7	3/4	20	17.1	16.0	14.1	11.5	8.3
		30	17.1	16.0	14.1	11.5	8.3
		40	15.0	14.1	12.6	11.5	8.3
		50	10.0	9.25	7.75	6.5	5.1
JS-10	1	20	20.5	18.0	15.5	12.5	9.3
		30	20.5	18.0	15.5	12.5	9.3
		40	20.3	17.8	15.3	12.5	9.3
		50	19.5	17.5	14.3	12.1	8.6

Note: Electrical service requirements in Part 5 of this manual (all require 220/230 volt service).

Checking Design Pump Size

Once your "Perc-Rite"™ System design has been calculated and all pressure/head losses and requirements have been determined for each of the system components you will then add all the pressure requirements together to a total required pressure for your "Perc-Rite"™ System. This is done as described in the summary of calculations section.

Now use Chart 4A to determine pump size requirement or to decide if you may need to make any "Perc-Rite"™ design layout changes to work within a particular performance capability of your pump size. Chart 4A is very simple to use.

After you have determined the total operating pressure requirement of the "Perc-Rite"™ System design, find the model number of the "Perc-Rite"™ pump you are using or you may check against the standard pump. The Model OJS-50 is the standard pump for W-20 models. Use the discharge pressure line for that pump model which is shown on the chart in 10 psi increments. You should always round up to the nearest increment of ten. If the operating pressure requirement of your system has been determined to be 27 psi, then use the 30 psi discharge pressure column to determine the capacity of your pump. You will then follow that discharge pressure column over to the column under the suction lift capacity which corresponds to the suction lift measured in feet for your system design. As discussed earlier in the suction lift section of calculating pressure losses, you will have rounded this number up to the nearest multiple of 5. This will then correspond to the suction lift columns in Chart 4A. The number in the box which corresponds to both the discharge pressure and the suction lift for that model pump will be the pumping capacity of that pump in gallons per minute.

You will have already calculated the total operating flow required for your "Perc-Rite"™ System design as discussed previously in this design manual. Compare the total operating flow requirement of your "Perc-Rite"™ System design against the pump capacity found for your pump model in Chart 4A. The pump capacity as found in Chart 4A must be equal to or greater than the total flow requirement for your "Perc-Rite"™ System design. If Chart 4A has indicated the capacity of your pump is equal to or greater than your flow requirement then you are assured your design is correct to assure proper functioning and dosing of your "Perc-Rite"™ System.

Example: The total operating pressure requirement of the "Perc-Rite"™ System is 41.5 psi based on calculations summarized in the previous section of this manual.

The suction lift is 10 feet based on calculations summarized in the previous section of this manual.

The OJS-50 standard pump is used for comparison. Chart 4A shows the OJS-50 at 41.5 psi discharge pressure and 10 feet suction lift to have a capacity of 8.8 GPM (since 43 psi is considerably less than the 50 psi column in Chart 4A, we may use 40 psi.)

You have previously calculated the total operating flow of your "Perc-Rite"™ design as shown in previous sections. The previous example has shown a system with 2,000 linear feet of dripper line with an operating flow of 18 GPM. Therefore, for this system design the capacity of the standard OJS-50 pump, at 43 psi and 10 feet lift and 8.8 GPM capacity as shown in Chart 4A, will not be sufficient. The OJS-50 will be 9.2 GPM short of the requirement. (18 GPM - 8.8 GPM = 9.2 GPM) Had the pump capacity been equal to or greater than the 18 GPM total flow requirement of the system, then your design is correct using the standard OJS-50 pump. You could then proceed with specifying this design and proceeding with the installation. This would be a simple layout of the drip absorption field in which you have already designed and calculated, without any modifications to the standard "Perc-Rite"™ W-20 unit, as shown in Figure 9 (see Page 52).

Design Modifications to meet Performance Specifications

This section will outline and discuss some of the design modification options available in order to accommodate pump and/or absorption field performance requirements.

Pump Specifications and Modifications

When it has been determined that a specific "Perc-Rite"™ design has a total flow requirement and pressure requirement that exceeds the performance Chart 4A for any one of the available pumps you have chosen, you may change your pump specification to another model number that may meet the performance requirements of your system design. This is very often one of the easiest ways to modify a standard system.

Example: Meeting design requirements by specification of different pumps. Using the previous example of a 2,000 feet absorption field requiring 18 GPM at a 43 psi calculated pressure requirement at 10 feet suction lift, take these requirement numbers to Chart 4A. You will find that no available "Perc-Rite"™ pump will supply the necessary pump capacity flow of 18 GPM at 40 psi and 10 feet of suction lift.

So other design modifications will be necessary to meet the performance requirements of this "Perc-Rite"™ System. These modifications are discussed in the next section.

First let us look at two more pump requirement situations using Chart 4A to size the "Perc-Rite"™ pump correctly.

1. Assume a system design requires a total operating flow of 13.5 GPM and a 29 psi pressure requirement and a 10 feet suction lift. Compare this to Chart 4A for the standard OJS-50 pump. (Since requirement is 29 psi, we will use 30 psi on the chart.) At 30 psi, the OJS-50 standard pump will supply 14.5 GPM at 10 feet suction lift. This pump meets the 13.5 GPM operating flow requirement of the "Perc-Rite"™ System design. Therefore, no design modifications are necessary.

2. Assume a system design requires a total operating flow of 18 GPM and a 30 psi pressure requirement and a 10 feet suction lift according to all calculations done according to this manual. Therefore, looking at Chart 4A, the standard OJS-50 pump falls short of 18 GPM by 3.5 GPM. However, by specifying the OJS-75 pump, you will be able to deliver 18.8 GPM at 30 psi at 10 feet of lift. Therefore, you may proceed with the design by changing the standard pump and specifying the 3/4 horsepower OJS-75 pump.

Please Note: If your system design can not be modified by any modification means discussed in this section to match the performance capabilities of any pump shown on Chart 4A, call Waste Water Systems, Inc. at 1-800-828-9045 for pump specification and design.

Modification of Design Layout

(Multiple Zone Absorption Fields)

Other than changing pump specifications, the most likely method of meeting the performance requirements of your "Perc-Rite"™ System design will be to split your absorption field into two zones that field flush separately, thus reducing the pressure and flow requirements of the system. Another alternative in systems that also require large pump capacities when dosing the absorption field for waste water disposal is to design the absorption field zones to dose and flush separately to adequately reduce your flow and pressure requirements. You will often encounter situations that will require either separate field flushing or separate field dosing and flushing to meet performance requirements.

Example: Separate field flushing

Let us assume a system design that requires 3,800 square feet of absorption area using 1,900 linear feet of dripper line. This system will require 9.7 GPM while dosing the absorption field.

Let us also assume all of your design has called for six dripper line connections to the flush manifold after running your laterals or loops on the contours. This system will require 9.6

GPM for flushing. Therefore, your total operating flow requirement is 9.6 + 9.7 = 19.3 GPM.

Let us assume your pressure requirement calculations show that the system will require 40 psi. (All the methods for calculating these pressure requirements have been described in previous sections of this manual.) Your suction lift has been determined to be 10 feet. Chart 4A shows that at 19.3 GPM and 40 psi and 10 feet of suction lift that there is not an available pump meeting the flow requirements of this design. However, by using two zones to flush the absorption field separately, you will do the following:

Let us say you have two equally sized zones that flush independently. Each zone will then have three dripper line flush connections at 1.6 GPM each. Therefore, the flushing flow requirement is then reduced to 3 x 1.6 = 4.8 GPM; therefore, 4.8 + 9.78 GPM (the absorption field dosing flow) would then be an operating total flow requirement of 14.5 GPM instead of the original 19.3 GPM when using a single zone. Therefore, by recalculating pressure requirements at this reduced flow, the total pressure required will also drop to approximately 33 psi. Now at 33 psi or even as high as 40 psi, with 10 feet of suction lift and 14.5 GPM flow requirement the OJS-75 3/4 horsepower will operate this system with separate flush zones. For an example layout, see Diagram in Figure 10 (see Page 53). Be sure when designing and ordering a "Perc-Rite"™ unit for the two zone flushing, you specify it to be programmed for that function as it will require special assembly and one extra field control valve. You will also notice in Figure 10 that two check valves will be required in each return flush line before reaching the common return pipe. This is to insure that while flushing each zone flushes separately by stopping any back flow of flush water into the other zone (PVC flapper type check valves).

Example: Separate flushing and absorption field dosing.

Let us assume a system design that requires a total of 6,000 square feet of disposal area using 3,000 linear feet of dripper tubing in the absorption field. This system will require 15.25 GPM while dosing a 3,000 linear feet absorption field.

Let us assume your design has allowed for eight dripper line connections to the return flush manifold. (Each lateral loop will be less than 400 feet long.) 8 x 1.6 GPM = 12.8 GPM Therefore, this design calls for 12.8 GPM to flush the entire absorption field. Your total operating flow is then 12.8 + 15.25 = 28.05 GPM. Upon reviewing all the pressure requirement calculations, you can tell that the total flow exceeds the "Perc-Rite"™ W-20 Model capabilities at 28.05 GPM. Therefore, you can already plan for this design to be a two zone system. (The limits of the pump Chart 4A and the filter unit Chart 2A do not even reach 28 GPM.) Now design this as a two zone system with separate flush and field dosing. Let us assume we can divide the system into two equal zones of 1,500 linear feet each. Therefore, your total operating flow per zone is now 7.6 GPM dosing and four connections to flush line x 1.6 GPM (6.4 GPM) for a total of 14 GPM per zone. At 14 GPM it is now within the performance of the "Perc-Rite"™ System. You can now continue with your design and specifications by using a two zone system. A diagram of a two zone system is shown in Figure 10 (see Page 53) and Figure 11 (see Page 54). (Some examples of a complete design computation for two zone systems will be shown later in this manual for your reference.)

Design of Unequally Sized Absorption Fields

Because of site limitations it may become necessary to layout the absorption field design for the "Perc-Rite"™ System in unequally sized areas. The considerations required when doing this are discussed in this section.

Unequally Sized Absorption Fields Dosed as One Zone

When an absorption field has been designed in two or more unequally sized areas but these areas are dosed by a single line (dosed at the same time) as shown in the diagram in Figure 9 (see Page 52), no special considerations will need to be addressed for this type of design. Since all the absorption field zones are being dosed at the same time, each zone will only accept the amount of waste water from that dosing cycle that it was designed to receive. This is because the flow rate for a smaller zone will be proportionally less than the larger zone. Also unequally sized zones or absorption fields that have been designed to flush separately but still dose via a single supply line at the same time will not need any special considerations in these designs, since only the field flushing is done separately, these fields will be dosed at the same time; thereby receiving the amount of waste water from the dosing cycle it was designed to receive.

Unequally Sized Absorption Zones Dosed Separately

When it becomes necessary to design a "Perc-Rite"™ System that doses two absorption fields of unequal size separately, it is in order to stay within the performance characteristics you desire with larger sized absorption fields. Therefore, when a system has been designed and laid out in this fashion your concerns that the system will mechanically perform correctly have already been addressed, you have done all your calculations to be sure of that. However, there is a second concern: **dosing the separate absorption systems correctly**. Since you are not going to dose the entire system at the same time but in two separate zones, you will need to be sure that each separate field area receives the correct amount of waste water and will stay within the loading rate limits you have established in your design. Each zone will have a different flow rate while dosing since they have different amounts of dripper line in them and different size absorption areas; therefore, they can not be dosed equal amounts of water. For example, if you design a single dose at 50 gallons and one absorption field zone is 1/3 smaller in size than the other absorption field zone then the amount of water that is dosed to that zone should be 1/3 less or about 33 gallons. If the amount dosed is the same for each field of unequal size then the smaller field will receive more than its share of the total daily flow that it was designed to handle.

Achieving equal dosing to different sized absorption fields that dose independently is made simple by using the "Perc-Rite"™ System. The "Perc-Rite"™ System controller has the capability to do this. This is one of the major improvements in the "Perc-Rite"™ System over all types of conventional systems.

Time Operated Dosing

Time dosing for waste water disposal is a major factor in contributing to keeping a soil absorption system functioning properly. It has already been proven that dosing cycles throughout a 24 hour period to dispose of waste water in a soil absorption system is the best way to keep the soil from over-saturation and soil clogging or failure. The instantaneous waste water application rate of the system should not exceed the water absorption capacity of the surrounding soil, to guard against surfacing or ponding of the effluent. This is difficult to calculate and achieve because water is applied at discrete points throughout the drip absorption field, so even if the total application rate is low, water could surface at some locations of the field. In all cases the gross application rate should be kept below the soil absorption rate. Dosing the absorption field in pulses instead of continuously or <u>on demand</u> as waste water flows into the dosing tank will help to avoid the over saturation problem. This is especially important when the design application rate is the same as the soil absorption rate. The instantaneous absorption capability of the surrounding soil varies with time. The absorption rate will normally be high at the beginning of the dosing cycle, before the surrounding soil saturates, then it gradually will reduce. By dosing in timed cycles with a <u>predetermined</u> amount of effluent waste water and then resting the soil absorption field you will keep the absorption rate of the surrounding soil at a higher value and reduce the risk of failure (ponding, surfacing, etc.).

It has been established that a very good guideline to follow in dosing absorption fields is to distribute the total daily waste water flow in six equal doses over a 24 hour period. This is not difficult using the "Perc-Rite"™ System, even with unequally sized absorption fields that separately dose.

How to Set Up Dosing with the "Perc-Rite"™ System

The set up of time dosing with a predetermined amount of waste water is a simple function of total daily flow divided by the number of doses per day and the flow rate of your absorption field.

Example of Time Dosing Fields that Supply at the Same Time:

Since a single absorption field or multiple absorption field zones that dose at the same time and not separately do not require special considerations to insure equal dosing, the formula to set up dosing is as follows:

$$\frac{\text{Amount of daily waste water flow}}{\text{Number of Doses per Day}} = \text{Amount of Each Dose}$$

Assume a 360 gallons waste water flow:

$$\frac{\text{360 GPD}}{\text{6 Doses/Day}} = \text{60 Gallons per Dose}$$

Important: Always order dose amounts in one gallon increments not in fractions.

Example: A calculated dose of 58.5 gallons should be programmed as 59 gallons.

The "Perc-Rite"™ controller will then be programmed to dose every 4 hours (6 times per day) at 60 gallons per dose. All "Perc-Rite"™ controllers are preset at the factory to dose every 4 hours since it has been established that this is a good guideline to dose and rest soils. So then the only variable that will be programmed into any "Perc-Rite"™ System of this type will be the amount of each dose in gallons. The system designer will designate this dose amount to the "Perc-Rite"™ dealer or manufacturer before the system is shipped to the jobsite. As in the above example for a 360 gallon per day system dosing the fields at the same time, you would specify a 60 gallon dosing cycle every 4 hours.

Example of Dosing Separate Fields of Equal Size

Dosing separate fields of equal size is basically the same as in single dosed fields. However, it must be decided if both fields will dose in the same 4 hour time cycle or in separate 4 hour time cycles. This probably will be a factor of the dosing flow rate of each field and the amount of time each cycle will run. Let us assume we have two equal fields with 1,100 linear feet of dripper line in each field. The flow rate for each field while dosing is 5.6 GPM or:

$$\frac{1{,}100}{2} = 550 \text{ Emitters} \times .61 = 336 \qquad \frac{336}{60} = 5.6 \text{ GPM}$$

The daily flow of this system is 360 gallons in 6 doses at 60 gallons each dose. Therefore, at 5.6 GPM per field, one field of 5.6 GPM will run approximately 10.7 minutes then in this example, the first of the equal absorption fields will dose 60 gallons in approximately 10.7 minutes and 4 hours later the next equal size field will dose for 10.7 minutes. Then another 4 hours later, the first field will dose again and so on. This scenario is favorable in that each field will be "at rest" for 8 hours between doses.

Important: Always try to keep each 4 hour dosing cycle a minimum of 6 minutes and a maximum of 12 minutes each.

<u>Example of Equal Sized Fields Dosing Separate</u>

450 GPD Flow with two equal fields of 1,200 L.F. each

$$\frac{1{,}200 \text{ L.F.}}{2} = 600 \text{ Emitters} \times .61 \text{ GPH} = 366 \ \frac{366}{60} = 6.1 \text{ GPM}$$

6.1 GPM Field Dosing Flow

$$\frac{450 \text{ GPD}}{6 \text{ Doses}} = 75 \text{ Gallons per Dose} \qquad \frac{75 \text{ Gallons}}{6.1 \text{ GPM}} = 12.2 \text{ Min. per Dose in Each Field Sep.}$$

In this case, it would be prudent to reduce the dose time to be under 12 minutes by dosing as follows:

Order the "Perc-Rite"™ unit to dose <u>both</u> fields at each 4 hour cycle to distribute the total 75 gallons. Each field will then dose approximately 38 gallons <u>separately but during the same time cycle</u>. Therefore, each dose cycle will dose each field as follows: 38.0 gallons at 6.1 GPM = 6.1 minutes each field during same dose cycle. After 4 hours the next dose cycle will be the same. Dosing each field separately to dispose of the desired 75 gallons per dose each and every 4 hour cycle.

<u>Dosing Unequal Absorption Fields Separately</u>

A little more calculation is required to set up the dosing on unequally sized absorption fields that dose separately. Since each absorption field is unequal in size, each field will have a different flow rate while dosing and a different amount of the daily flow to be attributed to disposal in that particular field or zone. Therefore, when your daily flow has been calculated and established for that particular system, it must be divided proportionately to each field that doses separately according to size.

Example: 450 Gallons per Day flow with two separately dosed fields of 1,100 linear feet and 700 linear feet each. Divide the daily flow proportionately as follows:

To find what percentage of the daily flow will go to each zone you will need to know the ratio of difference between each field size. The formula to do so is as follows: Take the number in feet of the total in either field one (1,100 feet) or field two (700 feet) and divide that number by the total footage amount in both fields. That will tell you what percentage of the daily flow will need to be dosed to that particular field.

Field One:

1,100 Linear Feet
----------------- = .61 or 61%
1,800 L.F. (both fields)

61% of the total daily flow of 450 gallons will be dosed to Field One.

450 gallons x .61 = 275 gallons.

The remaining amount of daily flow will go to Field Two.

Total Flow	450 Gallons per Day or 100%
Field One Flow	- 275 Gallons per Day or 61%
Field Two Flow	175 Gallons per Day or 39%

We have now established that in this example the 450 gallons per day will be dosed as follows:

Field One	1,100 Linear Feet	275 Gallons per Day
Field Two	700 Linear Feet	175 Gallons per Day

The next step will be to determine whether each field will be dosed during the same 4 hour dosing cycle or every other 4 hour dosing cycle (which allows each field to rest 8 hours). To determine this you must calculate the dosing flow rate for each field.

$$\text{Field One} \quad \frac{1{,}100\ \text{L.F.}}{2} = 550\ \text{Emitters} \times .61 = 335\ \frac{335}{60\ \text{Min.}} = 5.6\ \text{GPM}$$

$$\text{Field Two} \quad \frac{700\ \text{L.F.}}{2} = 350\ \text{Emitters} \times .61 = 214\ \frac{214}{60\ \text{Min.}} = 3.6\ \text{GPM}$$

Therefore,

$$\text{Field One} \quad \frac{275\ \text{GPD}}{5.6\ \text{GPM}} = 49.1\ \text{Minutes} \quad \text{Dosing Time Required}$$

$$\text{Field Two} \quad \frac{175\ \text{GPD}}{3.6\ \text{GPM}} = 48.6\ \text{Minutes} \quad \text{Dosing Time Required}$$

We have now established the total dosing time necessary per day for each field to dispose of the daily flow equally. We also know we have a maximum of 6 doses per day and a minimum of 3 doses per day per field if each field doses on separate 4 hour cycles. We also know that we wish to keep each dosing cycle between 6 and 12 minutes. Both Field One and Field Two should be dosed 6 times per day to stay under 12 minutes per dose.

$$\text{Field One} \quad \frac{49.1\ \text{Minutes}}{6\ \text{Doses}} = 8.2\ \text{Minutes}$$

$$\text{Field Two} \quad \frac{48.6\ \text{Minutes}}{6\ \text{Doses}} = 8.1\ \text{Minutes}$$

Therefore, this "Perc-Rite"™ System with unequal fields dosing separately should be ordered with both fields dosing during the same time cycle every 4 hours. Field One will dose 46 gallons in approximately 8.2 minutes then Field Two will dose separately but during the same dosing cycle (every 4 hours) 30 gallons in approximately 8.1 minutes.

As you can see, if each field were not dosed at every 4 hour dosing cycle, then each field will have only three dosing cycles per day. Field One would run for over 16 minutes per dose and Field Two would run for over 16 minutes per dose.

In some cases with larger fields and higher flow rates, you may be able to dose the unequal size fields at separate dosing cycles or every 8 hours (3 doses per field) because the increased flow rate for a larger field will keep your run time under 12 minutes per field. Below is another example.

Dosing Unequal Absorption Fields Separately (cont.)

Example: Assume a daily flow rate of 400 GPD. A low loading rate system has required you to design two separate fields. Field One has 1,500 linear feet of dripper line and Field Two has 2,000 linear feet of dripper line. To determine the proportional daily flow to each field is as follows:

A total of 400 GPD daily flow. The percentage to the larger field would then be calculated as:

$$\frac{2{,}000 \text{ Linear Feet}}{3{,}500 \text{ Linear Feet}} = .57 \text{ or } 57\%$$

Therefore, 57% of the total flow will go to Field Two which has 2,000 linear feet of dripper line. Field One, which has 1,500 linear feet, would be dosed the remaining flow of 43%.

400 GPD x .57 = 228 Gallons per Day

Therefore, 228 Gallons per Day to Field Two and 172 Gallons per Day to Field One.

Determine the flow rates during dosing:

$$\text{Field One} \quad \frac{1{,}500 \text{ L.F.}}{2} = 750 \text{ Emitters} \times .61 = 458 \quad \frac{458}{60 \text{ Min.}} = 7.6 \text{ GPM}$$

$$\text{Field Two} \quad \frac{2{,}000 \text{ L.F.}}{2} = 1{,}000 \text{ Emitters} \times .61 = 610 \quad \frac{610}{60 \text{ Min.}} = 10.2 \text{ GPM}$$

Determine the dosing time required:

$$\text{Field One} \quad \frac{172 \text{ GPD}}{7.6 \text{ GPM}} = 22.6 \text{ Minutes per Day}$$

$$\text{Field Two} \quad \frac{228 \text{ GPD}}{10.2 \text{ GPM}} = 22.3 \text{ Minutes per Day}$$

In this case, Field One will need to run 22.6 minutes per day and Field Two will need to run 22.3 minutes per day. This "Perc-Rite"™ System may be ordered to dose each field separately at separate 4 hour dosing cycles, allowing for 8 hour "rest" between field dosing.

22.6 Minutes / 3 Doses/Day	=	7.5 Minutes	22.3 Minutes / 3 Doses/Day	=	7.4 Minutes

Even though each field doses on separate 4 hour cycles only 3 times each per day, the run time is well within the 6 to 12 minute recommended run time range per cycle.

Dosing Volume and Pump Float Switch

After the maximum dosing volume that will be necessary for any dosing cycle has been determined, your design should also include the minimum depth of draw down in your dosing tank. This depth should provide a volume of waste water that is equal to or greater than the maximum amount necessary for each dosing cycle. Therefore, you will need to know the volume of water per inch in depth of the dosing tank intended for use in your design.

Example: Suppose your standard 1,000 gallon dosing tank has a volume of 15 gallons per inch of water depth. Assume you have two fields that dose separately and the larger field requires at least 60 gallons per dosing cycle. Therefore, your float switch must be set with enough tether or swing length to pull down 4" of water so:

$$\frac{\text{60 gallon dose}}{\text{15 gallon per inch}} = 4''$$

All "Perc-Rite"™ pump float switches will allow for an even ratio of 1" draw down for every 1" of tether length. Therefore, in this situation you should allow at least 4 inch of swing length in the float switch tether.

Another Example: Dosing tank with 10 gallons per inch of water depth. Dosing volume required per dose is 50 gallons.

$$\frac{\text{50 gallon dose}}{\text{10 gallons per inch}} = 5''$$

So set the float switch with 5 inch of swing.

Note: You may wish to always set the float switch with an extra inch or so swing. the extra volume allowed will not upset the dosing balance of your "Perc-Rite"™ System as the system will be set to only dose the amount specified for each field.

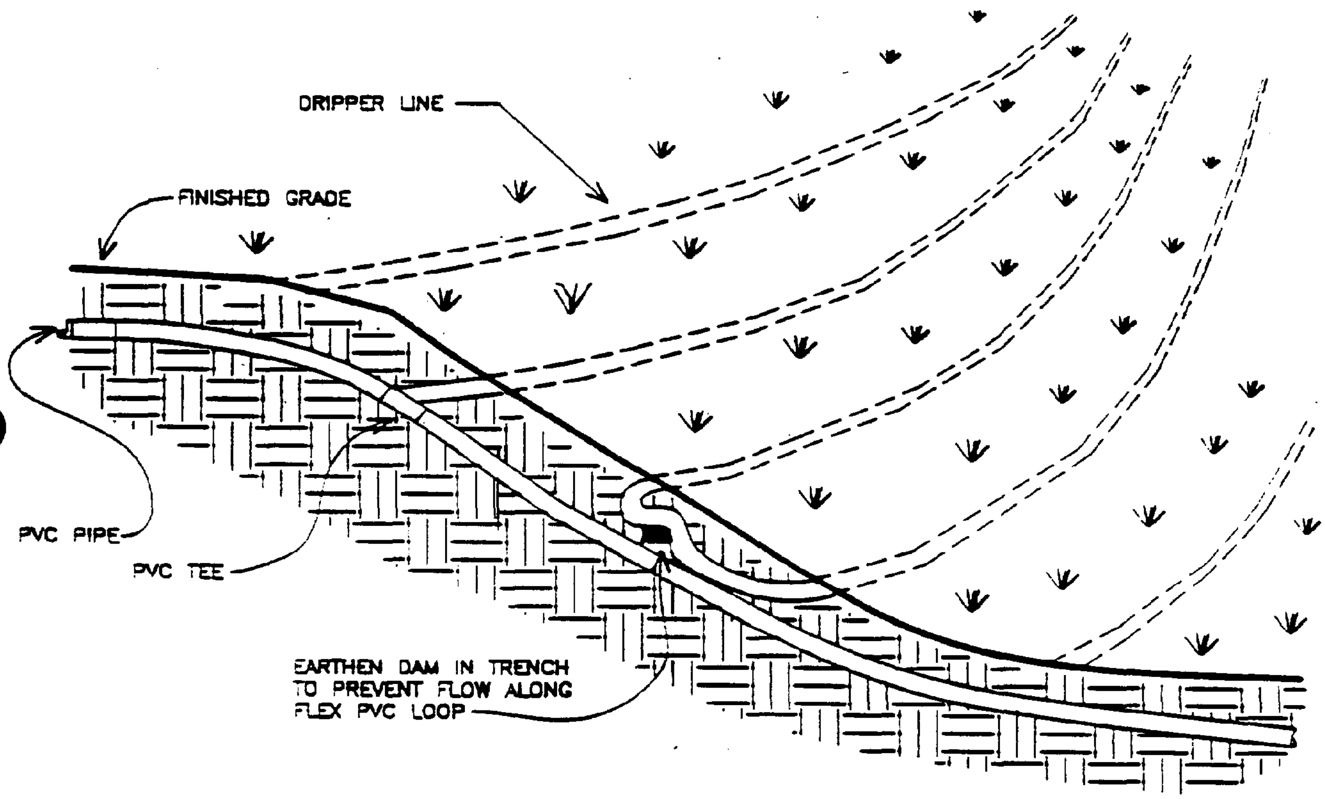
Summary

Time dosing of the "Perc-Rite"™ Drip Soil Absorption System is an integral part of each individual system design. Some designs such as multiple absorption field designs and separate dosing absorption fields will require more calculations than other "Perc-Rite"™ designs. The method of calculations to determine these dosing variables have been described in this section. The variable time and flow dosing capability of each individual "Perc-Rite"™ System is one of the advantages of using such a system for a long term soil absorption system.

"Perc-Rite"™ Designs on Sloping Ground

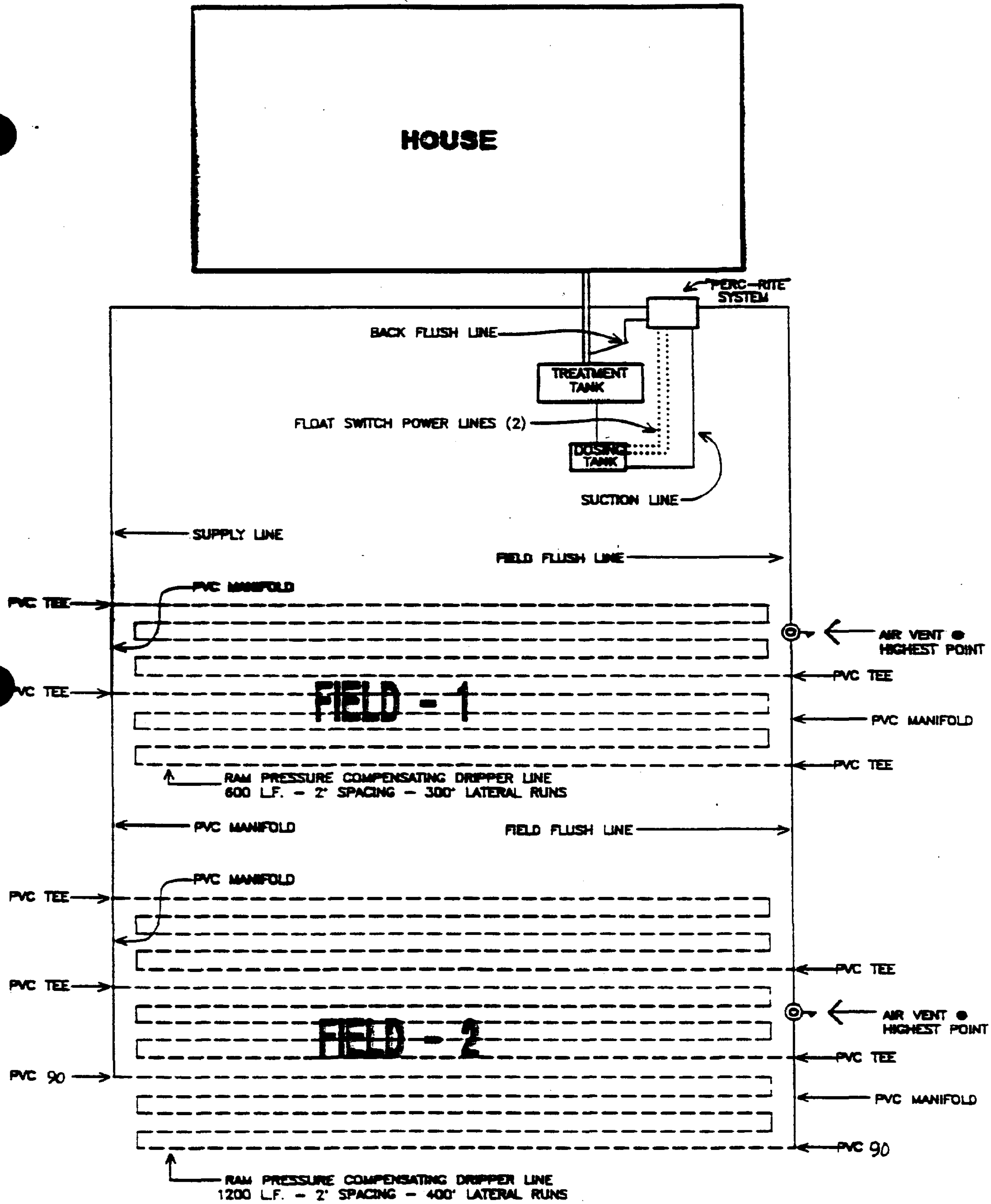
As described previously, the pressure compensating type dripper line used in the "Perc-Rite"™ Waste Water Disposal System provides for even drip emission rates throughout a broad range of pressure differences. The problem with most pressurized soil absorption systems is, on sloping ground the pressure differences caused by head losses in the vertical lift of such systems would cause problems in providing equal effluent distribution. Since these head losses do not effect the "Perc-Rite"™ Systems dripper discharge rates, no special design considerations will need to be calculated. Simply follow the guidelines for determining pressure requirements as described previously in this manual. As long as you design within those performance standards, no additional elevation change head losses will need to be calculated. This is another advantage "Perc-Rite"™ Systems have over other conventional pressure systems or non-pressure compensating turbulent flow type drip systems.

When designing "Perc-Rite"™ Systems on severe slopes of 20% or more because of installation difficulties and excess gravitational pull on the effluent down hill it may be advantageous to separate the dripper line laterals runs to 36 inches separation instead of the normal 24 inches separation. This can be a design decision, or based on soil scientist recommendation and personal preference based on any difficulties that may be encountered installing laterals as close as 24 inches apart on severe slopes. In very poor soils with slow absorption rates, you may increase absorption field sizing when separating the dripper laterals to 36 inches apart. This is to assure that you do not exceed the instantaneous loading rate capabilities of that particular soil, because you have increased the run time of each dose by using less dripper line in the absorption area required by your design. Remember that all lateral lines must be placed on the ground contours to keep each lateral dripper line as close to level as possible. When making loops or turns with connections to the supply line, always use solid tubing, not dripper line, in the turns as described and shown previously in this manual. When you make a turn to the next elevation, you may even wish to raise the solid tubing in your loop or turn over an earthen dam. This will ensure that the water from the upper lateral dripper line will not follow the solid tubing down to the next lower lateral line, see Diagram below.



SECTION – DRIPPER LINE TO FOLLOW CONTOURS
& PVC MANIFOLD PERPENDICULAR TO GRADE

FIGURE 9



HOUSE
"PERC-RITE" SYSTEM
BACK FLUSH LINE
TREATMENT TANK
FLOAT SWITCH POWER LINES (2)
DOSING TANK
SUCTION LINE
SUPPLY LINE
FIELD FLUSH LINE
PVC MANIFOLD
PVC TEE
AIR VENT @ HIGHEST POINT
PVC TEE
PVC TEE
FIELD - 1
PVC MANIFOLD
PVC TEE
RAM PRESSURE COMPENSATING DRIPPER LINE
600 L.F. - 2' SPACING - 300' LATERAL RUNS
PVC MANIFOLD
FIELD FLUSH LINE
PVC MANIFOLD
PVC TEE
PVC TEE
PVC TEE
AIR VENT @ HIGHEST POINT
FIELD - 2
PVC TEE
PVC 90
PVC MANIFOLD
PVC 90
RAM PRESSURE COMPENSATING DRIPPER LINE
1200 L.F. - 2' SPACING - 400' LATERAL RUNS

FIGURE 10

SEPARATE SUPPLY AND COMMON RETURN
W/ CHECK VALVES ALLOWING INDIVIDUAL
FIELD FLUSHING

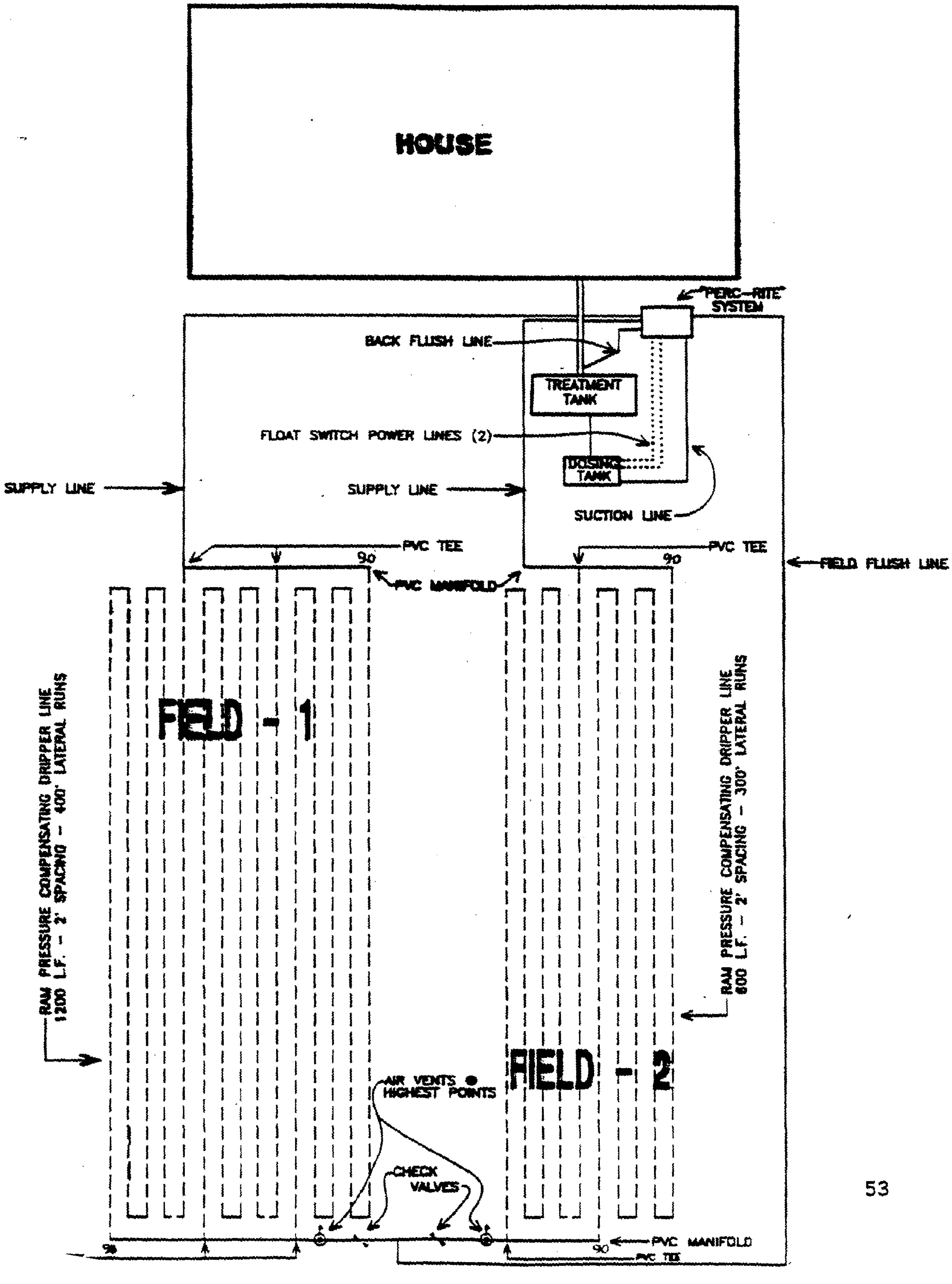
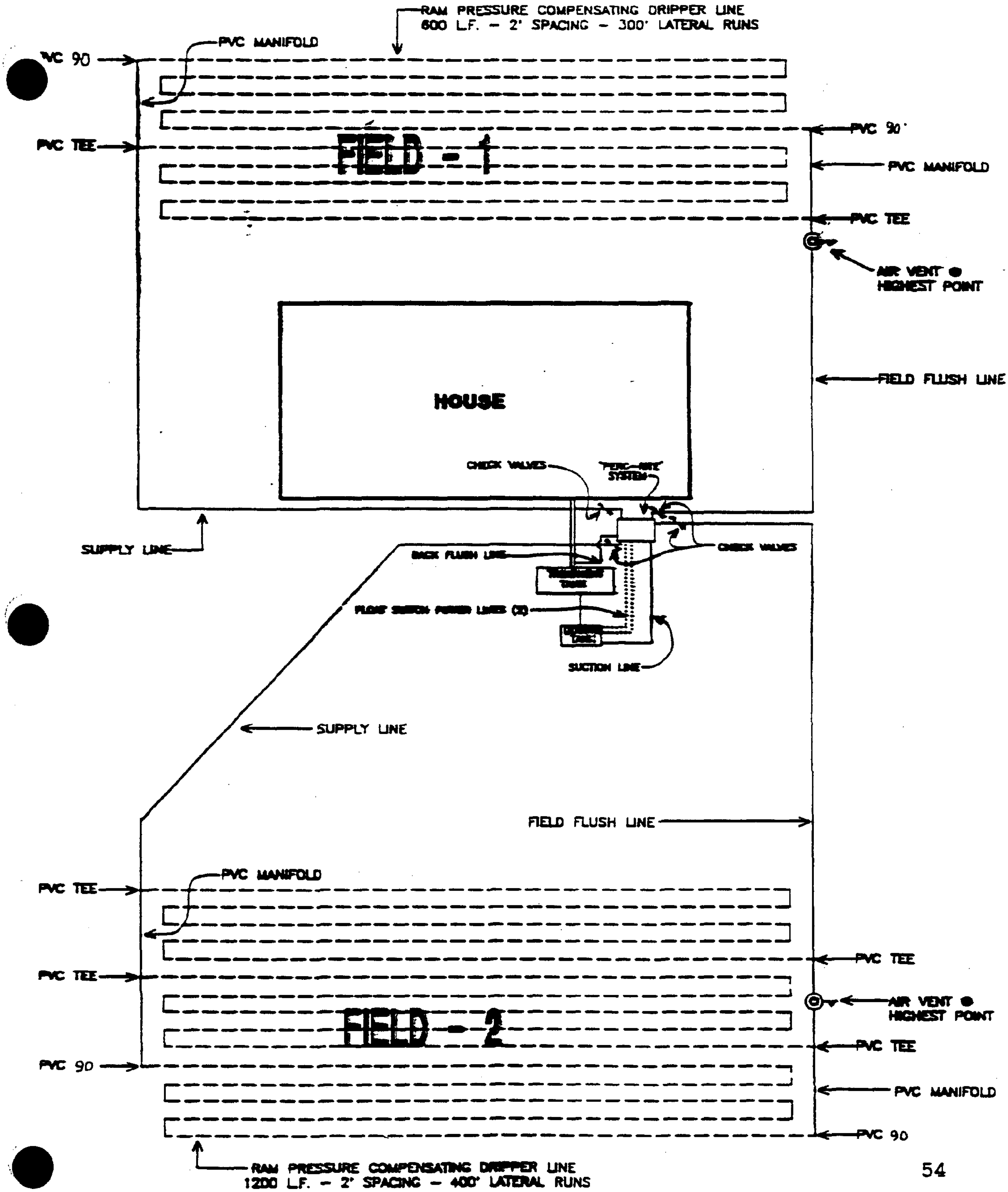


FIGURE 11

SEPARATE SUPPLY AND SEPARATE RETURN LINES



EXAMPLE DESIGN CALCULATIONS

FOR "PERC-RITE"™ SYSTEMS

USING THIS MANUAL

Example Design 1: (See Example Worksheet on Page 68)

Daily Flow - (Line 2 on Worksheet) A three bedroom house x 150 GPD/bedroom = 450 GPD

Loading Rate - Soil reports show Sandy Clay Loam. (Line 3 on Worksheet) Chart 1 shows .15 gallons per square feet per day as a loading rate. (Line 4 on Worksheet)

Total Area Requirement - (Line 5 on Worksheet)

$$\frac{\text{450 GPD}}{\text{.15 Loading Rate}} = \text{3,000 Sq. Ft.}$$

Total Dripper Line Required - (Line 7 on Worksheet)

$$\frac{\text{3,000 Sq. Ft.}}{\text{2 Feet Spacing}} = \text{1,500 Linear Feet}$$

Determine Layout and Shape - (Line 13 on Worksheet) Area on the lot is approximately 60 feet x 50 feet or 3,000 Sq. Ft. This is conducive to five laterals or loops 300 feet long. (5 connections to the return flush line.)

Assumptions: Distance to drip field is 110 feet (1" PVC). Distance from dosing tank to pump is 50 feet (1 1/4" PVC). Suction elevation lift is 10 feet. Return flush line length is 120 feet (1" PVC).

Septic tank size - a 1,000 gallon septic tank (Line 9 on Worksheet) and a 1,000 gallon dosing tank (twice the daily flow) (Line 10 on Worksheet).

Depth of Line - (Line 11 on Worksheet) Soils analysis shows optimum depth at 18" deep.

Begin Design

Dosing Flow Rate - (Line 14a on Worksheet)

$$\frac{\text{1,500 L.F.}}{\text{2' Spacing}} = \text{750 Emitters x .61 GPH} = 457.5 \qquad \frac{457.5}{\text{60 Min.}} = \text{7.6 GPM}$$

7.6 GPM dosing rate

Field Flush Flow Rate - 5 connections to the return flush line x 1.6 GPM = 8 GPM Flushing Rate (Line 14c on Worksheet)

Total flow requirement 15.6 GPM

Calculate Flow Loss and Pressure Requirement

A. Suction Line Loss - (Line 15a on Worksheet) Chart 1A is 1 1/4" at 15.6 GPM = 3.05 Loss per 100 feet.

$$3.05 \times .5 = 1.53 \qquad \frac{1.53}{2.31} = .7 \text{ psi (Round up to 1 psi)}$$

B. Suction Lift - (Line 15a on Worksheet) 10 Vertical feet

C. Supply Line - (Line 15b on Worksheet) 1" 110 feet long. Chart 1A shows 1" PVC at 15.6 GPM = 11.8 per 100 feet.

$$11.8 \times 1.1 = 12.98 \qquad \frac{12.98}{2.31} = 5.6 \text{ psi (Round up to 6 psi)}$$

D. Return Flush - (Line 15c on Worksheet) 1" 120 feet long. 1" PVC at 8 GPM = 3.68 per 100 feet.

$$3.68 \times 1.2 = 4.41 \qquad \frac{4.41}{2.31} = 1.9 \text{ psi (Round up to 2 psi)}$$

E. Pump Filter Unit - (Line 16 on Worksheet) Chart 2A shows 15.6 GPM = 6.85 psi

F. Fittings - 0

G. Elevation Changes (assume) - (Line 17 on Worksheet) 5 vertical feet to the highest point in dripper field

$$\frac{5}{2.31} = 2.16 \text{ psi (Round up to 2.2 psi)}$$

H. Dripper Line Laterals - (Line 18 on Worksheet) Chart 3A 300 feet = 15.2 psi

Summary:

A.	Suction Line - (Line 15a on Worksheet)	1.0 psi
B.	Suction Lift of 10 feet	
C.	Supply Line - (Line 15b on Worksheet)	6.0 psi
D.	Return Flush - (Line 15c on Worksheet)	2.0 psi
E.	Filter Unit - (Line 16 on Worksheet)	6.85 psi
F.	Fittings	0.0 psi
G.	Elevation Changes - (Line 17 on Worksheet)	2.2 psi
H.	Dripper Line Laterals - (Line 18 on Worksheet)	15.2 psi
	Total Pressure Req. - (Line 19 on Worksheet)	33.25 psi

Check Pump Performance

On Chart 4A use 40 psi at 15.6 GPM at 10 feet lift. The standard OJS-50 will not work. The 3/4 horsepower pumps will not work. The JS-10 1 horsepower pump will work at this performance requirement. It provides 17.8 GPM at 40 psi at 10 feet of lift. You could use this design as it is and specify this available pump. However, instead of specifying the 1 horsepower pump, you may want to make some modifications to your design. For example:

You may increase the force main supply and return flush line size to 1 1/4". The reduction in supply and return loss brings your total pressure requirement down to less than 30 psi. You may then specify the OJS-75 or the JS-7 3/4 horsepower pumps.

In order to use the standard pump and reduce the performance requirement even more you may split the field to flush separately. By reducing the flushing flow requirement to a maximum of 3 connections to the flush line in each separate flush zone you will now reduce the total flow requirement by 2 connections x 1.6 GPM = 3.2 GPM. This will reduce all the pressure requirements as calculated above and you may now use a total flow of 12.4 GPM filter unit pressure loss is now down to 5.0 psi according to Chart 2A. Your supply line losses and return line losses will also reduce.

As you can see, all the methods for calculating performance requirements may be used to modify any "Perc-Rite"™ design to meet your specific requirements.

The next step in reducing the performance requirements of this system would be to split the absorption field into two zones that flush <u>and</u> dose separately. This should reduce the pressure and flow requirement significantly. Let us work through the same example after modifying the design into separate field flushing and dosing with 1 1/4" supply and return lines.

Two zones: 900 feet and 600 feet each

Dosing Flow -

Field One $\dfrac{900 \text{ L.F.}}{2}$ = 450 Emitters x .61 = 275 $\dfrac{275}{60 \text{ Min.}}$ = 4.57 GPM

Field Two $\dfrac{600 \text{ L.F.}}{2}$ = 300 Emitters x .61 = 183 $\dfrac{183}{60 \text{ Min.}}$ = 3.05 GPM

Flushing Flow - Use Field One, which has 3 flush line connections, to determine flush flow 3 x 1.6 = 4.8 GPM.

Total Operating Flow - 4.8 GPM + 4.5 GPM (largest field) = 9.3 GPM Total Flow. Use 9.3 GPM to determine all pressure requirements.

Pressure Requirements -

Suction Line - (Line 15a on Worksheet)	.3 psi
Suction Lift of 10 feet	
Supply Line - (Line 15b on Worksheet)	1.0 psi
Return Flush - (Line 15c on Worksheet)	1.0 psi
Filter Unit - (Line 16 on Worksheet)	3.0 psi
Fittings	0.0 psi
Elevation Changes - (Line 17 on Worksheet)	2.2 psi
Dripper Line Laterals - (Line 18 on Worksheet)	15.2 psi
Total Pressure Req. - (Line 19 on Worksheet)	22.7 psi

Performance Requirement - For 23 psi at 9.3 GPM and 10 feet of lift will now allow you to use the standard pump which will more than supply the required head and pressure.

Now let us look at dosing. 450 GPD.

$$\text{Field One} \quad \frac{900 \text{ Feet}}{1{,}500 \text{ Feet}} = .60 \text{ or } 60\%$$

Therefore, Field One gets 60% of total daily flow (450 x .60 = 270 GPD)

Field One	270 GPD
Field Two	180 GPD
	450 GPD

$$\text{Field One} \quad \frac{270 \text{ GPD}}{4.5 \text{ GPM}} = \text{60 Min Daily Dose Time} \qquad \frac{60 \text{ Min.}}{6 \text{ Doses}} = \text{10 Min per Dose}$$

$$\text{Field Two} \quad \frac{180 \text{ GPD}}{3.0 \text{ GPM}} = \text{60 Min Daily Dose Time} \qquad \frac{60 \text{ Min.}}{6 \text{ Doses}} = \text{10 Min per Dose}$$

Note: By dosing six times per day or every 4 hours, the 10 minutes per dose falls within the recommended 6 to 12 minute dosing time.

Therefore, order the "Perc-Rite"™ unit for each separate zone to dose in the same 4 hour dose cycle, six times per day. (45 gallons for Field One and 30 gallons for Field Two.)

Pump Float Switch

Since the dosing volume per cycle will be 45 gallons in this case (largest field) the float switch is to be set as follows:

Assume a 1,000 gallon dosing tank with 15 gallons volume per inch of water depth.

$$\frac{45 \text{ gallons}}{15} = 3 \text{ inches}$$

Specify float switch to be set at a minimum of a 3 inch swing.

Example Design 2:

Daily Flow - (Line 2 on Worksheet) A four bedroom house x 150 GPD/bedroom = 600 GPD

Loading Rate - Soil reports show Clay Soil (Line 3 on Worksheet). Lot size allows us to use a .1 gallons per square feet per day loading rate (Line 4 on Worksheet).

Total Area Requirement - (Line 5 on Worksheet)

$$\frac{600 \text{ GPD}}{.1 \text{ Loading Rate}} = 6{,}000 \text{ Sq. Ft.}$$

Total Dripper Line Required - (Line 7 on Worksheet)

$$\frac{6{,}000 \text{ Sq. Ft.}}{2 \text{ Feet Spacing}} = 3{,}000 \text{ Linear Feet}$$

Determine Layout and Shape - (Line 13 on Worksheet) Assume two areas on lot located 4,000 sq. ft. in one area and 2,000 sq. ft. in the second area.

With such a large system design, you should already consider this a two zone system.

Field One is a 4,000 sq. ft. area will allow for 2,000 linear feet of dripper line in six laterals or loops, the longest will be 380 feet.

Field Two is a 2,000 sq. ft. area will allow for 1,000 linear feet with four lateral runs, the longest will be 300 feet.

Septic tank size - a 1,500 gallon septic tank (Line 9 on Worksheet) and a 1,000 gallon dosing tank (Line 10 on Worksheet).

Assumptions: The distance to the dripper field from the unit is 100 feet. The distance from the dosing tank to the pump is 50 feet. The return flush line distance is 100 feet.

Since this is a large system with a large flow rate, you can already assume we should use larger pipe sizes. Use 1 1/4" for the suction, supply line and return flush.

Begin Design

Dosing Flow Rate - (Line 14a on Worksheet)

One $\frac{2{,}000 \text{ L.F.}}{2' \text{ Spacing}} = 1000 \text{ Emitters} \times .61 \text{ GPH} = 610 \quad \frac{610}{60 \text{ Min.}} = 10.1 \text{ GPM}$

Two $\frac{1{,}000 \text{ L.F.}}{2' \text{ Spacing}} = 500 \text{ Emitters} \times .61 \text{ GPH} = 305 \quad \frac{305}{60 \text{ Min.}} = 5.0 \text{ GPM}$

Use 12.7 flow rate in pressure requirement calculations.

Field Flush Flow Rate - (Line 14c on Worksheet)

Field One - 6 connections x 1.6 GPM = 9.6 GPM

Field Two - 4 connections x 1.6 GPM = 6.4 GPM

Use 9.6 GPM in pressure/flow requirement calculations.

Total flow requirement 9.6 + 10.1 = 19.7 GPM (Round to 20 GPM)

Calculate Flow Loss and Pressure Requirement

A. Suction Line Loss - (Line 15a on Worksheet) Chart 1A is 1 1/4" at 20 GPM = 5.21 Loss per 100 feet.

$$5.21 \times .5 = 2.60 \qquad \frac{2.60}{2.31} = 1 \text{ psi}$$

B. Suction Lift - (Line 15a on Worksheet) 5 Vertical feet

C. Supply Line - (Line 15b on Worksheet) 1 1/4" at 19 GPM. Use 5.21 per 100 feet.

$$\frac{5.21}{2.31} = 2.2 \text{ psi}$$

D. Return Flush - (Line 15c on Worksheet) 1 1/4" at 9.6 GPM

1.44 per 100 feet.

$$\frac{1.44}{2.31} = .6 \text{ psi (Round to 1 psi)}$$

E. Pump Filter Unit - (Line 16 on Worksheet) Chart 2A shows 20 GPM = 12.25 psi

F. Fittings - 0

G. Elevation Changes (assume) - (Line 17 on Worksheet)

Elevation rise from pump unit to the highest point in the field is 5 feet.

$$\frac{5 \text{ feet}}{2.31} = 2.16 \text{ psi}$$

H. Dripper Line Laterals - (Line 18 on Worksheet) Chart 3A 380 feet = 23.1 psi

Summary:

A.	Suction Line - (Line 15a on Worksheet)	1.0 psi
B.	Suction Lift of 5 feet	
C.	Supply Line - (Line 15b on Worksheet)	2.2 psi
D.	Return Flush - (Line 15c on Worksheet)	1.0 psi
E.	Filter Unit - (Line 16 on Worksheet)	12.25 psi
F.	Fittings	0.0 psi
G.	Elevation Changes - (Line 17 on Worksheet)	2.16 psi
H.	Dripper Line Laterals - (Line 18 on Worksheet)	23.1 psi
	Total Pressure Req. - (Line 19 on Worksheet)	41.71 psi

<u>Check Pump Performance</u>

Chart 4A

Use 40 psi at 5 feet suction lift. The OJS-100 and the JS-10 1 horsepower pumps will supply over 20 GPM at 5 feet of suction lift and 40 psi. However, this is the upper limits of the "Perc-Rite"™ W-20 System. when large designs meet this threshold great care should be given to the calculations and installations. The system has been designed with separate dosing and flushing so further performance requirement reductions by anymore design changes will be minimal. You may wish to call Waste Water Systems, Inc. at 1-800-828-9045 to check on specifying another pump.

Time Dosing

$$\text{Field One} \quad \frac{2{,}000 \text{ L.F.}}{3{,}000 \text{ L.F.}} = .66$$

66% of the daily flow is to be dosed to Field One.

.66 x 600 GPD = 396 gallons per day

Field Two gets the remaining flow of 204 gallons per day.

Field Dosing Flows

Field One 10 GPM
Field Two 6.4 GPM
Total Dosing Time per Day

$$\text{Field One} \quad \frac{396}{10} = 39.6 \text{ minutes}$$

$$\text{Field Two} \quad \frac{204}{6.4} = 31.8 \text{ minutes}$$

Six doses per day each field.

$$\text{Field One} \quad \frac{39.6}{6} = 6.6 \text{ minutes} = 66 \text{ gallons}$$

$$\text{Field Two} \quad \frac{31.8}{6} = 5.3 \text{ minutes} = 34 \text{ gallons}$$

You should probably order this unit to dose both fields during every 4 hour dosing cycle. However, you have the flexibility in the "Perc-Rite"™ unit to dose one of these zones every other dosing time cycle or every 8 hours. This may be desirable in the smaller zone which is now running only 5.3 minutes. The field dosing would then be as follows:

Field One - 6 doses per day at 66 gallons or 6.6 minutes per dose as shown above.

Field Two - 3 doses per day (every other dosing cycle) or

$$\frac{204 \text{ gallons}}{3 \text{ doses}} = 68 \text{ gallons per dose}$$

with a run time of 10.6 minutes each dose. This field will then dose every 8 hours.

So the ideal way to design and order the "Perc-Rite"™ in this case would be by dosing zone one 66 gallons per dose, every dosing cycle (6 times each day) and dose zone two at 68 gallons per dose at every other dosing cycle (8 hours) three times per day.

Pump Float Switch

Since the dosing volume per cycle will be 68 gallons in this case (largest field) the float switch is to be set as follows:

Assume a 1,000 gallon dosing tank with 15 gallons volume per inch of water depth.

$$\frac{68 \text{ gallons}}{15} = 4.6 \text{ inches}$$

Specify float switch to be set at a minimum of a 5 inch swing.

Designs not Meeting Performance Capabilities of the "Perc-Rite"™ W-20 System

In some cases with larger flows and very low soil loading rates you will be unable to meet the performance capabilities of the "Perc-Rite"™ System as shown in this manual, even by using all the design modifications detailed in this manual. This does not mean a "Perc-Rite"™ System will not work in that particular situation. You should call Waste Water Systems, Inc. at 1-800-828-9045 for "Perc-Rite"™ Equipment Specifications that will meet those performance requirements. Waste Water Systems, Inc. will provide a system with modifications to the Standard Available Equipment to meet your design specifications, which may include a larger "Perc-Rite"™ Filtration and Pump Unit.

PART FIVE

Other Specifications and Equipment Used in the Complete "Perc-Rite"™ System

All the equipment necessary to complete a "Perc-Rite"™ Waste Water Disposal System should be included into a single worksheet that can be used to be sure the system will be installed as designed. The worksheet can also serve as a checklist to be sure the design was done correctly and has taken into consideration all the required calculations. A sample of a worksheet is shown at the end of this section (see Page 68). A copy of this worksheet and an accurate sketch of the system design layout including all drainage and landscaping requirements should be filled out and drawn up for every system designed. Copies of the worksheet and sketch should be given to the local health department or regulatory agency. These worksheets and sketches are also helpful for the installation contractor in making a list of materials necessary.

Septic Tank and Dosing Tank

As noted earlier a "Perc-Rite"™ System has at least two separate tanks. A septic tank and dosing tank or any specified pretreatment means such as an aerobic treatment unit with a dosing tank are what is usually installed. If the "Perc-Rite"™ System is being used to replace an existing septic system the existing septic tank may be used (after being pumped out and cleaned) and only one additional tank will need to be installed.

The septic tank receives waste water directly from the house. It is sized according to state and local regulations for conventional systems. The septic tank should be of two-compartment design for maximum solids retention. It is very important that the septic tank and pumping chamber are watertight. One-piece tanks are best. When using two-piece tanks, the tongue-in-groove joint must be carefully sealed with asphalt rope mastic. This is very important. The tanks (especially the dosing tank) must be water tight.

Effluent from the two-compartment septic tank flows by gravity through a four inch solid PVC pipe to the pumping chamber. The pumping chamber should have a liquid capacity of at least two times the daily waste flow from the house and can be a single-compartment design. Dosing tanks may be sized to hold only 24 hours daily flow capacity. However, it is beneficial to allow for more storage in case of equipment failure or power outage. The dosing tank must be provided with aboveground concrete or masonry (or their equivalent) manhole risers to provide easy access for clean-out and pump service. The riser should be placed over the primary chamber of the septic tank and above the pump access hole in the pumping chamber. Risers should be wide enough to accommodate the existing lids on the tanks, should extend at least six inches above the finished grade of the site and should also be covered with a concrete, metal or plastic lid to local specifications.

Standard well tiles can be used for the risers, provided that the inside diameter is larger than the access hole in the tank. All joints must be sealed to prevent the infiltration of surface runoff and ground water to the tanks.

Pipes and Fittings

All pipe and fittings in a "Perc-Rite"™ System should be made of PVC plastic. PVC is lightweight, easy to use and resists corrosion. All joints must be sealed with an appropriate PVC solvent cement. In instances where check valves are necessary, PVC flapper type check valves may be used.

Air Vents

As discussed previously in this design manual, air vent/vacuum breakers are required at the high point of every dripper field or zone. Waste Water Systems, Inc. will supply air vents as needed. They should be installed in a valve box. Plastic valve boxes of any choice are fine. The air vents are fitted with 1/2" male pipe threads that fit standard 1/2" female adapters.

Electrical Service

The electrical requirement for any W-20 "Perc-Rite"™ residential unit is as follows:

For any available "Perc-Rite"™ pump specify a dedicated 220/230 volt 20 amp service. (3 wire service)

Home Water Saving Devices

Any home with a "Perc-Rite"™ System should minimize the hydraulic load on the soil absorption system by using low flow water saving devices such as low flow showerheads and low flow commodes. These devices are a simple low cost way of reducing water use without inconveniencing the homeowner.

<u>Site Preparation Specifications</u>

One of the most important concerns for a "Perc-Rite"™ System is to protect the site from soil disturbance by heavy equipment. Removal or compaction of the topsoil, especially during wet weather, may destroy the site's suitability for a "Perc-Rite"™. As soon as the absorption area has been designated, it should be flagged, roped off and "quarantined" from construction traffic. No site preparation or LPP construction work should occur if the soil is wet. As a rule of thumb, if the soil is too wet to plow, it is too wet to disturb for system construction.

After the location is staked out and the soil is dry enough to plow, the site should be cleared of brush and small trees. If larger trees are removed, they should be cut off rather than uprooted in order to avoid creating depressions and damaging the soil-pore network.

Provisions must be made for intercepting or diverting surface water and shallow ground water away from the absorption area, septic tank and pumping chamber. This can be done with grassy swales, open ditches or curtain drains.

If the site requires imported fill to improve surface drainage, it must be incorporated evenly into the underlying natural soil. It is very important that no sharp interface remain between the natural and imported soil layers. Before applying the imported fill to the absorption area, the ground surface must be tilled with a small plow or cultivator. Fill should be applied with a minimum of wheeled traffic on the area, and the area tilled again to ensure even mixing. A very small tractor should be used to spread the material around and to provide a convex shape to the area. There should be no low spots or depressions, and the final shape should shed, rather than accumulate rainwater. Use of fill to supplement the soil profile is discussed in Part 6 of this manual.

After the area has been cleared and shaped, the location of the lateral lines and supply manifold should be accurately staked out according to design specifications. Each lateral line should be laid out along a contour. One lateral may be higher or lower than the next one, but each individual lateral run should follow a contour evenly. In no case should a lateral line be allowed to slope away from the manifold in any direction without using earthen dams or other measures as shown in the Diagram on Page 12.

Final Landscaping

After a "Perc-Rite"™ System has been installed, the following should be specified in final landscaping requirements to ensure the system will not be overloaded with excess rain water and runoff.

- The distribution field is shaped to shed rain water and is free of low areas.
- Curtain drains, grassy swales or ditches for diverting ground and surface water are properly installed.
- Gutter and downspout drains are directed away from the system.

Finally, the entire area should be planted with grass in order to prevent erosion. The soil should be properly tilled, limed (if necessary) and fertilized before planting. After applying an appropriate grass seed, the area should be heavily mulched with straw or other suitable material.

DESIGN WORKSHEET FOR "PERC-RITE"™ SPECIFICATIONS

1. HOUSE SIZE ______ BEDROOMS _______ SQ. FT.
2. ______ GALLONS PER DAY (GPD)
3. SOIL CLASSIFICATION ________________________________
4. SOIL LOADING RATE _________________ GALLONS/SQ. FT./DAY
5. TOTAL ABSORPTION AREA REQ. (Line 2 ÷ Line 4) _________ SQ. FT.
6. DRIPPER LATERAL SPACING __________________ (STANDARD IS 2 FT.)
7. TOTAL DRIPPER LINE REQUIRED (Line 5 ÷ Line 6) _______________
8. DRIPPER LINE LATERAL LENGTH (LONGEST):
   _____ ZONE ONE _____ ZONE TWO _____ ZONE THREE _____ ZONE FOUR
9. SEPTIC TANK/ATU SIZE ____________ GALLONS
10. DOSING TANK SIZE _______ GALLONS _______ GAL/INCH IN DEPTH
11. DEPTH OF LINE _____________
12. ABSORPTION FIELD LAYOUT: (CHECK ONE)
    _____ ZONE ONE _____ ZONE TWO _____ ZONE THREE _____ ZONE FOUR
13. ABSORPTION FIELD ZONE SIZE:
    ZONE ONE ________ SQ. FT. ________ LINEAR FEET
    ZONE TWO ________ SQ. FT. ________ LINEAR FEET
    ZONE THREE ________ SQ. FT. ________ LINEAR FEET
    ZONE FOUR ________ SQ. FT. ________ LINEAR FEET
14. FLOW RATES: *USE THE LARGEST ZONE FOR THE DESIGN*
    a. DOSING FLOW RATE (GPM)
    _____ ZONE ONE _____ ZONE TWO _____ ZONE THREE _____ ZONE FOUR
    b. NUMBER OF RETURN FIELD FLUSH CONNECTIONS
    _____ ZONE ONE _____ ZONE TWO _____ ZONE THREE _____ ZONE FOUR
    c. FIELD FLUSH FLOW RATE (Line 14b X 1.6 GPM)*
    _____ ZONE ONE _____ ZONE TWO _____ ZONE THREE _____ ZONE FOUR
    d. TOTAL FLOW REQ. (Line 14a + Line 14c)*
    _____ ZONE ONE _____ ZONE TWO _____ ZONE THREE _____ ZONE FOUR
15. a. SUCTION LINE SIZE ______ INCHES AND LENGTH ______
    PRESSURE LOSSES (PSI) ______ SUCTION LINE LIFT ______
    b. FORCE MAIN SUPPLY LINE PIPE SIZE IN INCHES
    _____ ZONE ONE _____ ZONE TWO _____ ZONE THREE _____ ZONE FOUR
    FORCE MAIN SUPPLY LINE PIPE LENGTH IN FEET
    _____ ZONE ONE _____ ZONE TWO _____ ZONE THREE _____ ZONE FOUR
    PRESSURE LOSSES (PSI) ______
    c. RETURN FLUSH LINE PIPE SIZE IN INCHES
    _____ ZONE ONE _____ ZONE TWO _____ ZONE THREE _____ ZONE FOUR
    RETURN FLUSH LINE PIPE LENGTH IN FEET
    _____ ZONE ONE _____ ZONE TWO _____ ZONE THREE _____ ZONE FOUR
    PRESSURE LOSSES (PSI) ______
16. MODEL 20 PUMP AND FILTER LOSS (PSI) ______
17. ELEVATION RISE ______ PSI
18. DRIPPER LINE LATERAL LOSS ____ PSI
19. TOTAL PRESSURE REQ.:(TOTAL OF 15a-c+16+17+18 LOSSES) ______ PSI
20. PUMP CAPACITY ________ GPM AT ________ PSI AT ________ LIFT
21. PUMP MODEL # _____________ HORSEPOWER _________
22. TIME DOSING PER ZONE
    ONE ______GPM ______MIN/DOSE ______CYCLES ______GAL/DOSE
    TWO ______GPM ______MIN/DOSE ______CYCLES ______GAL/DOSE
    THREE ______GPM ______MIN/DOSE ______CYCLES ______GAL/DOSE
    FOUR ______GPM ______MIN/DOSE ______CYCLES ______GAL/DOSE
23. FLOAT SWITCH DRAW DOWN _________ INCHES _________ GALLONS
24. ELECTRICAL REQUIREMENT __________ VOLT __________ AMP
25. CHECK VALVES ON SEPARATE RETURN FLUSH LINES _____ YES _____ NO
26. LANDSCAPE MODIFICATIONS NEEDED ______ YES ______ NO

EXPLAIN:

LOW FLOW WATER SAVING DEVICES USED ______ YES ______ NO

PART SIX

Modified "Perc-Rite"™ Systems Using Fill

Most sites with a restrictive horizon or a seasonally high, water table within 24 inches of the surface are not suitable for a standard "Perc-Rite"™ System. Many are not suitable for any soil-absorption, waste-treatment system. But some of these sites can be used for waste treatment if the soil is supplemented with fill that has been carefully selected and added.

When there is approximately 24 inches of usable soil on an acceptable site, a modified "Perc-Rite"™ System using some fill may be designed. The existing soil must be of suitable or provisionally suitable texture, structure and permeability. After the addition of some fill in order to maintain 12 to 18 inches of cover over the dripper lines trenches may be placed as shallow as 3 inches into the natural soil. When there is less than 24 inches of usable soil a <u>mound type</u> system may be designed. The design and construction of mound systems will already have design criteria set by the local health departments which are used in other conventional soil absorption systems. The same criteria should be used when building a mound system for installation of a "Perc-Rite"™ Drip Soil Absorption Systems. Usually the fill material will be a soil with at least 25% course or medium sand and 50% fine or very fine sand (a Sandy Loam or Sandy Clay Loam type soil). Your soil scientist and local health department should be contacted regarding mound systems. Loading rates should always be determined on the most restrictive horizons under your fill or mound.

Design of Fill or Mound Systems

Other than the criteria described above, the only difference between designing a modified and standard "Perc-Rite"™ System is the calculation of the fill requirements. The volume of the fill necessary is the area to be filled multiplied by the depth of fill. The area to be filled is the absorption field plus a five foot buffer around the edges.

Step 1. Calculate the area to be filled. Add 10 feet to the length and width of absorption area to allow for the buffer space.

Example: For a 60 feet x 30 feet absorption field to be filled 1 foot deep:

Total area = 70 feet x 40 feet

Step 2. Calculate the volume of fill needed.

Example:

V fill = total area x depth of fill

V fill = 70 feet x 40 feet x 1 foot = 2,800 ft^3

Step 3. Convert to cubic yards.

Example:

$$V\ fill = \frac{2,800\ ft^3}{27\ ft^3\ per\ yd^3} = 104\ yd^3$$

The remaining design steps follow the procedure as described previously.

Installation

The success of a modified "Perc-Rite"™ System depends on the care used in selecting and incorporating the fill material. The fill must have a Sandy Loam or Loamy Sand texture. The fill should not be hauled or worked wet.

As with all "Perc-Rite"™ Systems, the site must be protected from traffic. Prior to incorporating the fill, brush and small trees should be removed and the soil surface loosened using a cultivator or garden plow. It is very important that the soil be worked only when dry. Working damp or wet soil can cause compaction and sealing, leading to failure of the system.

Fill is moved to the system using a front end loader, being careful to avoid driving on the plowed area. The first load of fill is pushed into place using a very small crawler tractor with a blade or a roto-tiller with a blade. The fill is then tilled into the first few inches of natural soil to create a gradual boundary between the two. Failure to do this could ruin the system by forming a barrier to water movement at the soil-fill interface. Subsequent loads of fill are placed on the system and tilled, until the desired height is reached. The site should be shaped to shed water and be free of low spots before proceeding.